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A JOURNAL OF HIGHWAY RESEARCH

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Volume 32

April 1962-February 1964

U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

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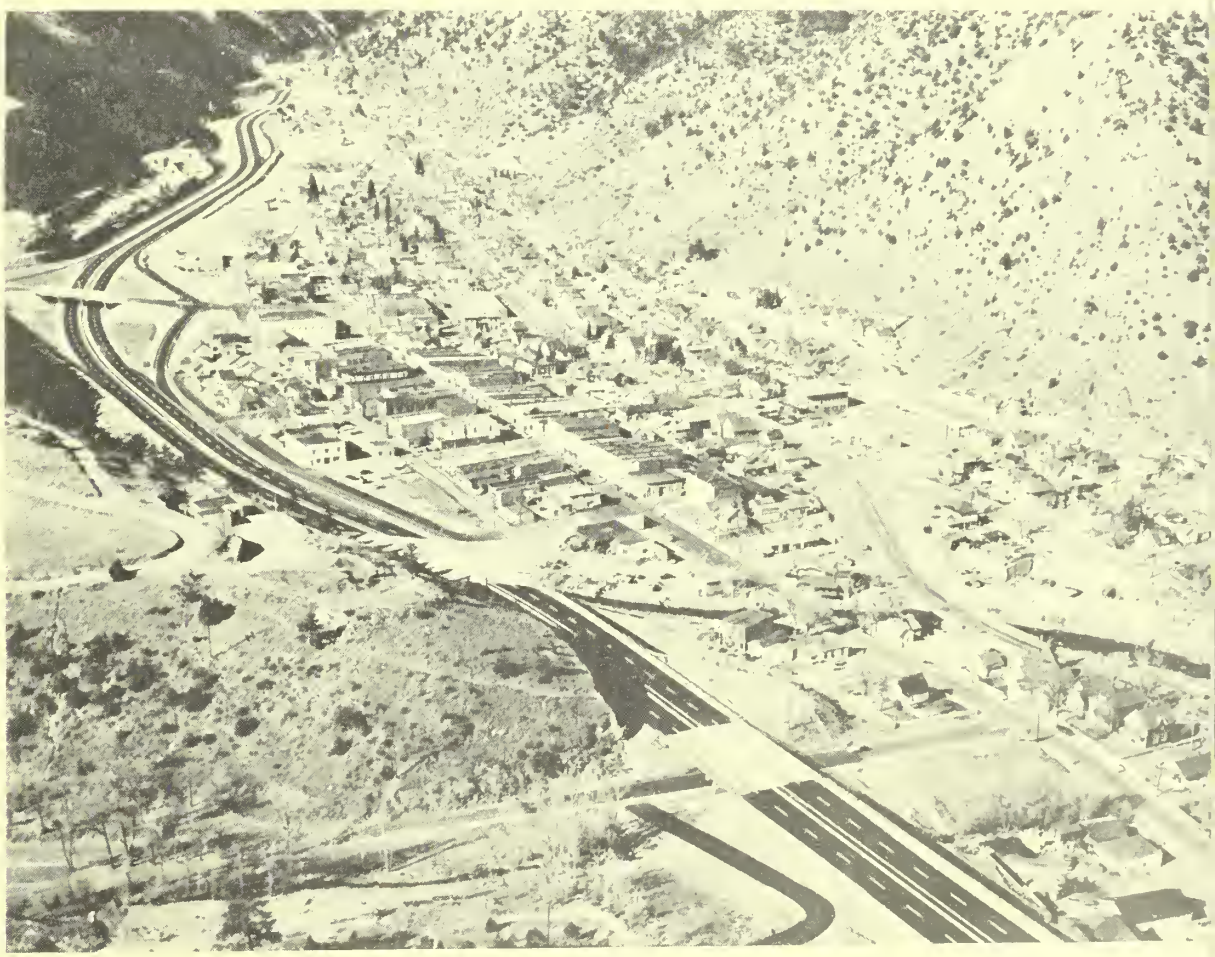

View of new location of Interstate Route 70 as it bypasses Idaho Springs, Colo., approximately 40 miles west of Denver.

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Muriel P. Worth, *Editor*

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The title sheet for vol. 31, Apr. 1960–Feb. 1962, of PUBLIC ROADS magazine is now available. This sheet contains a chronological list of article titles and an alphabetical list of authors' names. Copies of this title sheet can be obtained by a request to the editor of the magazine, Bureau of Public Roads, Washington 25, D.C.

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Social Effects of Modern Highway Transportation

BY THE HIGHWAY AND LAND ADMINISTRATION DIVISION
BUREAU OF PUBLIC ROADS

Reported [1] by FLOYD I. THIEL, Economist,
Economic Impact Research Branch

This article focuses attention on some of the social effects and changes that have accrued from construction of modern highways. Knowledge of these social effects—both beneficial and undesirable—should be of assistance to those concerned with planning highways, especially during public hearings concerning the routes and the construction of the highways. It appears fundamental that the social effects attributed to previously constructed highways should be considered when a new highway or relocation of an existing highway is being planned.

The requirement for public hearings on route location is a recognition of the need for consideration of highway effects, social as well as economic. At such hearings, socio-economic questions may loom large; for example, the likely effect of a highway relocation on schools, local government services, and residential areas may be of great concern to those that will be affected.

This article contains a general discussion of the influences that modern highways and their construction have had upon life in the United States, including those on residences in urban and rural areas; population mobility and the resulting developments in education, recreation, public services, and certain voluntary associations; and the availability of services for a mobile public.

Introduction

THE WIDESPREAD social effects of highway transportation are everywhere apparent. In the United States there now are more automobiles than households, and more adults holding drivers' licenses than library cards (*1*).[2] But it is, of course, much easier to determine that highway transportation is having far-reaching social effects than it is to identify and measure these effects with any precision. Social effects may be regarded as those influences that change the relationship between people and social institutions such as the family, community, government, schools, churches, etc.

Information gathered during various highway impact studies and other pertinent material [3] has been used in this article to direct attention to certain social changes or trends that can be associated with, or attributed to, highway improvements. Although sociologists may attribute additional effects to highways, this article contains discussions primarily of the highways' effects on: (1) Suburban development, (2) population mobility, (3) residences, (4) relocation of residences, (5) employment conditions, (6) travel time, (7) public services, (8) certain voluntary associations, (9) rural areas, (10) recreation, and (11) drive-ins and mobile services. In some cases these changes or effects overlap.

Although information contained in this article is of a general nature and does not permit definite conclusions as to the effects of modern highways, it is believed that the pervasive role of modern highway transportation in urban, suburban, and rural living is undeniable. Though there may be some question as to whether improved highway transportation has been the cause or the effect of some of the changes referred to in this article, it is clear that life in the United States without modern highway transportation would be far different from what it is today.

Social Effects

An opportunity appears to exist for improvement of public relations for the highway builders through the development and dissemination of information concerning the social effects of highways. For a number of reasons, the public relations problems faced by highway builders are sometimes especially vexing. Those opposing or questioning highway projects are often more vocal than those supporting or indifferent to the projects. The negative effects of highway construction (dust, noise, inconvenience, etc.) may be apparent immediately, and the positive effects may be delayed so that the negative influence is often overemphasized. Furthermore, objective information as to the social and economic effects of highway projects often is not available or is not disseminated to the public. As a researcher in Kansas put it recently, costs are largely in the present, and they tend to be relatively definite and measurable, while benefits are only future anticipations and tend to be relatively diffuse and unmeasurable.

Benefits

Not all the social effects of highways are beneficial. Certain developments that have been associated with highways—for example, noisy or littered drive-in facilities, poorly planned or administered parks for mobile homes, or ribbon developments—obviously have harmful effects. However, these developments may be economically beneficial to certain people, at least temporarily. While it must be recognized that highways have some harmful effects, no suggestion is intended that the harmful effects equal or approach the beneficial effects, even if quantitative measures were available for precise comparison. Any investment of the magnitude of that for the highway system of the United States is expected to provide substantial net benefits, both social and economic.

A good deal of the attention that has been focused on economic effects of highways has been useful in indicating what the social effects are. It seems safe to assume that increased residential land values, which are often associated with highway improvements, are a

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[1] Presented at the 41st annual meeting, Highway Research Board, Washington, D.C., January 1962.

[2] References indicated by italic numbers in parentheses are listed on p. 10.

[3] In the preparation of this paper, particularly the portion dealing with church activities, information gathered and summarized by Mr. Robert Daiute, an employee of the Bureau of Public Roads during the summer of 1961, was of considerable assistance.

reflection of certain social benefits—perhaps easier access to libraries, schools, government buildings, churches, friends' residences, and social contacts. For fairly small communities, 1,200 population or less, it is possible that bypass routes may be located in such a way that residential and commercial development may be somewhat restricted.

Selected Indicators of Highway Effects

All social changes have multiple causes and those occurring in areas of highway improvement cannot all be attributed to the effect of the highway, as might be suggested by the title of this article. In addition to the effect of the highway, the social and economic changes occurring along Massachusetts Route 128 have been attributed to a variety of causes: a pent-up need for suburban expansion, the abundant supply of technically trained personnel, and aggressive banking. Furthermore, the causes and effects of social trends interact so that generally it is difficult to distinguish between them. Highways may have important influences on family living, such as broadening employment opportunities and facilitating school and church consolidations. At the same time, changes in family composition—more children per family or more working wives—have certain social effects that should be considered in relation to highway building.

Nothing new on the use of quantitative terms to describe the social effects of highways is presented in this article. But as Professor Hennes has pointed out, the lack of precise instruments for measuring changes does not excuse highway people from exercising some measure of intuitive judgment in the determination of social benefits. To ignore realities because numbers cannot be found to put into formulas would be most unfortunate (*2*).

In the absence of summary quantitative indicators, perhaps some of the social effects of highways can be detected by careful observation and analysis of certain social changes or trends that appear to have been strongly influenced by highway transportation. Some indication of the highway's influence can be gathered by noting the timing of certain trends. In commenting on the adjustment that farmers make to a loss of land for highway right-of-way, Professor McKain has pointed out that highways may simply accelerate and smooth the way for changes that are inevitable, such as the consolidation of farm properties (*3*). It seems entirely possible that improved highways also may hasten other changes that would have occurred later without the highway. Changes or trends of this nature appear to include the whole suburban movement, population mobility, school and church consolidations, and generally expanded community boundaries, upgrading of residential property, improved public services, larger farms, fewer farm buildings, and more off-farm employment. Other possible indicators of highway effect include residential development, changes in employment conditions and in recreational activities, and the development of drive-ins and mobile services.

Suburban Development

The phenomenal increase in population of the United States metropolitan areas is well known. Since 1950, about 90 percent of the growth in population has occurred in metropolitan areas. Within metropolitan areas, the suburban fringes are experiencing the fastest growth rates—approximately seven times as fast as the rate of growth for central cities. By 1980, three-fourths of the Nation's 245 million people are expected to be urbanites.

The drift of population to metropolitan areas was underway in this country well before the advent of motor vehicular transportation and can hardly be attributed to improved highways. But the influence which improved highways are having on the current acceleration of the movement of people to the edges of metropolitan areas is undeniable. Of all the influences responsible for the phenomenal growth of the suburbs—mass-produced homes, shorter working hours, easy mortgage financing, septic tanks, and driven wells—improved highway transportation has surely been one of the most important. This highway influence on suburban development is quite similar to other effects that highways exert and is caused primarily by the increased accessibility of suburban areas and the easier driving that modern highways permit.

A good indication of the important role highways play in suburban growth is provided by Richardson, Tex., a community from which travel time to downtown Dallas was cut from more than 30 minutes to about 17 minutes, when the North Central Expressway was completed in 1955. Richardson's subsequent growth in terms of population increase and manufacturing activity was greater than that of comparable towns that were not influenced by the Expressway.

It often has been pointed out that the growth of a metropolitan organization has resulted from the conquest of distance, a barrier to community size. With modern highway transportation, it has become possible for almost any worker, regardless of economic or social level, to commute some distance to work. Reduction of the limiting effects of distance has permitted an extension of community boundaries of up to 35 miles or more from the center, approximately one hour's travel time.

Modern highway transportation has been instrumental in filling in many of the vacant areas near urban centers, as well as in extending the limits of settled areas. These underdeveloped areas had resulted from the pattern of development prior to the 1930's when, in most cities, development was feasible only if the site could be conveniently reached by public transportation. This emphasis on public transportation in major cities frequently resulted in a starlike growth pattern. The present tendency, for urban development to spread out more evenly and to fill in the gaps between public transportation arteries, has certain advantages—for example, the resultant improvement in the availability of utilities or public services. Thus, modern highways have the effect of increasing the supply of land available for urban development.

Population Mobility

It is obvious that the modern highway system has had an effect on the mobility of the American people. This mobility, of course, cannot be attributed solely to the effects of highway transportation. In any one year, about 7 percent of the American people move their place of residence across county lines. Disproportionately large numbers of these people are attracted to those areas without public transportation, which makes it necessary for them to depend on automobiles and highways for transportation (*4*). This tendency for people to move is a response to social and economic opportunities; it is purposeful at least from the viewpoint of those moving. Migration apparently occurs primarily for economic reasons. This mobility may have certain harmful effects in that family and community ties may be broken or weakened; but at the same time, opportunities for additional and desirable contacts may be opened up. In addition, mobility of people geographically usually speeds and eases their social mobility and reduces provincialism.

Residences

The suburban upsurge has depended greatly on the large-scale conversion of land to residential purposes. Single-family residences have become the largest consumer of land in urban areas, from 50 to 75 pecent of the total urbanized area. Thus, the extent of the expansion of urbanized areas—more than a million acres a year—is governed largely by the quantities of land converted to residential use.

Effects that modern highways have had on residential development are quite obvious. By shortening travel time, and in effect bringing more land close to employment and shopping centers, modern highway facilities make it feasible to develop land that formerly was too remote for residential development. The attraction that sites located near modern highways have for residential developers is so obvious and generally recognized that it hardly seems necessary to cite specific instances where new residential developments have been associated with highway improvements. The effect of modern highways on residential areas perhaps can be discerned more clearly by considering other aspects on which there is less general agreement. Some of these aspects are (1) the opinion of residents toward nearby highways, (2) whether highways attract a particular type of resident, and (3) how highways affect community blight.

Opinions of Residents

While attitudes are no objective measure of benefits or disadvantages, the opinions of residents about a nearby highway facility are an important indication of the social impact that the highway has on the local community. A highway must not only facilitate the transport of goods and people it must be generally acceptable to the residents of the area through which it passes

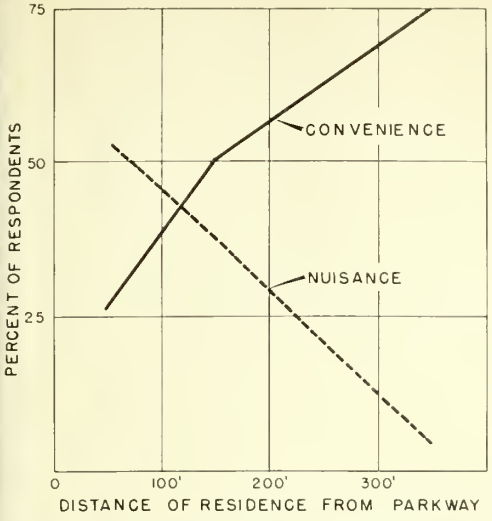

Figure 1.—Residents' opinions of Hutchison River and Bronx River Parkways (New York) by distance from parkway.

Without this acceptance, a highway can generate such harmful effects as lowered land values—land values to a great extent depend on how people feel about property and the surrounding area—and general community blight.

In the determination of attitudes toward highway improvements, careful attention must be given to the highway's stage of development. Typically, the degree of acceptance of a highway facility increases with time; perhaps because the positive effects from highway improvement sometimes are not immediately appreciated, and the apprehensions about possible negative highway effects often are not confirmed. In spite of this tendency for many residents to be initially suspicious of nearby highway improvements, modern highways appear to be quite acceptable to the major portion of the residents of areas served by these facilities. This acceptance is based, not only on the attitude that the highway is good for the community generally but, in many instances, on the individual's feeling that the highway provides benefits to him personally or to his property. There is also some evidence that certain community leaders display a more favorable attitude toward highway facilities than average citizens (*5*). A few of the findings from attitude surveys are referred to in the following paragraphs.

Favorable reaction

In Texas, about 80 percent of the residents questioned along the Dallas Central Expressway felt that the highway made their property more attractive and increased its value. Along a portion of one San Antonio expressway, about 60 percent of the owners said the attractiveness of their property had been enhanced as a result of the highway improvement, while about 40 percent felt that their property had not been affected. Along another section of a San Antonio expressway—Loop 13—the nearby residents, by a 2-to-1 ratio, believed the highway's effect to be beneficial.

In Baltimore, Md., residents near the Beltway who were questioned also considered their highway locations to be generally satisfactory, with the exception of those located near a particular interchange handling a considerable volume of truck traffic.

Attitude surveys in New York State also indicated that residents generally have found important traffic arteries in their neighborhoods to be acceptable, though the acceptance of a highway facility was found to depend largely on the highway's proximity. As might be expected, there is a relationship between the proximity of a person's residence to the highway and his attitude toward that highway. In Westchester County, for example, the percentage of residents with a favorable opinion of the nearby highway varied from about 33 percent in the first 100-foot zone to more than 75 percent in the 300- to 400-foot zone. About 50 percent of the residents in the first 100-foot zone regarded the highway facility as a nuisance, but only about 5 percent in the 300- to 400-foot zone regarded it so. This relationship is illustrated in figure 1.

The tendency for residents near, but not abutting, a highway facility to have a higher opinion of the facility than residents whose property abuts the highway has been noted in other studies, such as those made in Texas and California. Another finding from the New York studies, which agrees generally with results obtained from Texas opinion surveys, is that residents with children tend to object more to a nearby highway than residents without children, presumably because of the traffic hazard.

In addition to the general improvement in access to employment and shopping that highways provide, the general acceptance of highway facilities was sometimes based on an interesting combination of reasons. In San Antonio, benefits mentioned were the assistance the lighted highway provided in keeping prowlers away, the easier circulation of cooling breezes along the right-of-way, and the entertainment value provided by passing cars. Several of these advantages have been mentioned in studies in other States; for example, in Illinois advantages mentioned included the interest and activity along the highway, assured light and air, and the parklike environment pro-

Figure 2.—Varying uses of land near a highway, illustrated by the Gulf Freeway, Houston, Tex.

vided by the broad right-of-way. In New York, residents referred to the special advantage a highway location provides when they are directing friends and relatives to their homes.

While there appears to be a general acceptance of highway facilities by nearby residents, opposition is sometimes raised, particularly when a highway cuts through a residential community. For example, those opposing a particular highway location in Rhode Island stated their objections thusly: A 4-acre playground will be taken entirely for an interchange. Police and fire protection will be more costly because travel distances will be greater. Of 443 houses, 114 will be taken and 40 will be isolated, leaving the community one-third smaller. New friendships may develop, but more of the social life will have to be carried on by car. Many children, instead of having 15-minute walks, will have to ride a bus to school (*6*).

Highways and Community Development

A highway can have a significant influence on the general character of a community development—either to uplift it or to depress it, according to the type of residents it attracts. Analysis of results of several investigations suggests that the type of residential development and the residents that are being attracted to highway locations are tending to upgrade the communities involved. In Blairsville, Pa., residents arriving after completion of a new bypass route generally had more formal education, higher incomes, more interest in community affairs, and more upward occupational mobility than was the case for people with a longer period of residence (*5*). In Monroeville, Pa., where rapid development has been experienced along with greatly improved highway access, recent residential construction has been predominantly for white-collar employees, in contrast to the character of its earlier residential development. A similar upgrading in residential development has been noted in association with the Atlanta Expressway in Georgia. Caution obviously should be exercised against overstating the highway influence. Some increase in the proportion of technical and professional people attracted to any new development, with or without highways, is of course to be expected in view of the increasingly higher educational and training standards for the work force.

Highway effect on blight

Highways can exert either a beneficial influence on a community or can contribute to its blight. The noise and accident hazard associated with some highways, especially those with free access, can have a blighting influence on nearby residences. Even controlled-access highways can have such a blighting effect if residences are too close or are poorly oriented to the right-of-way.

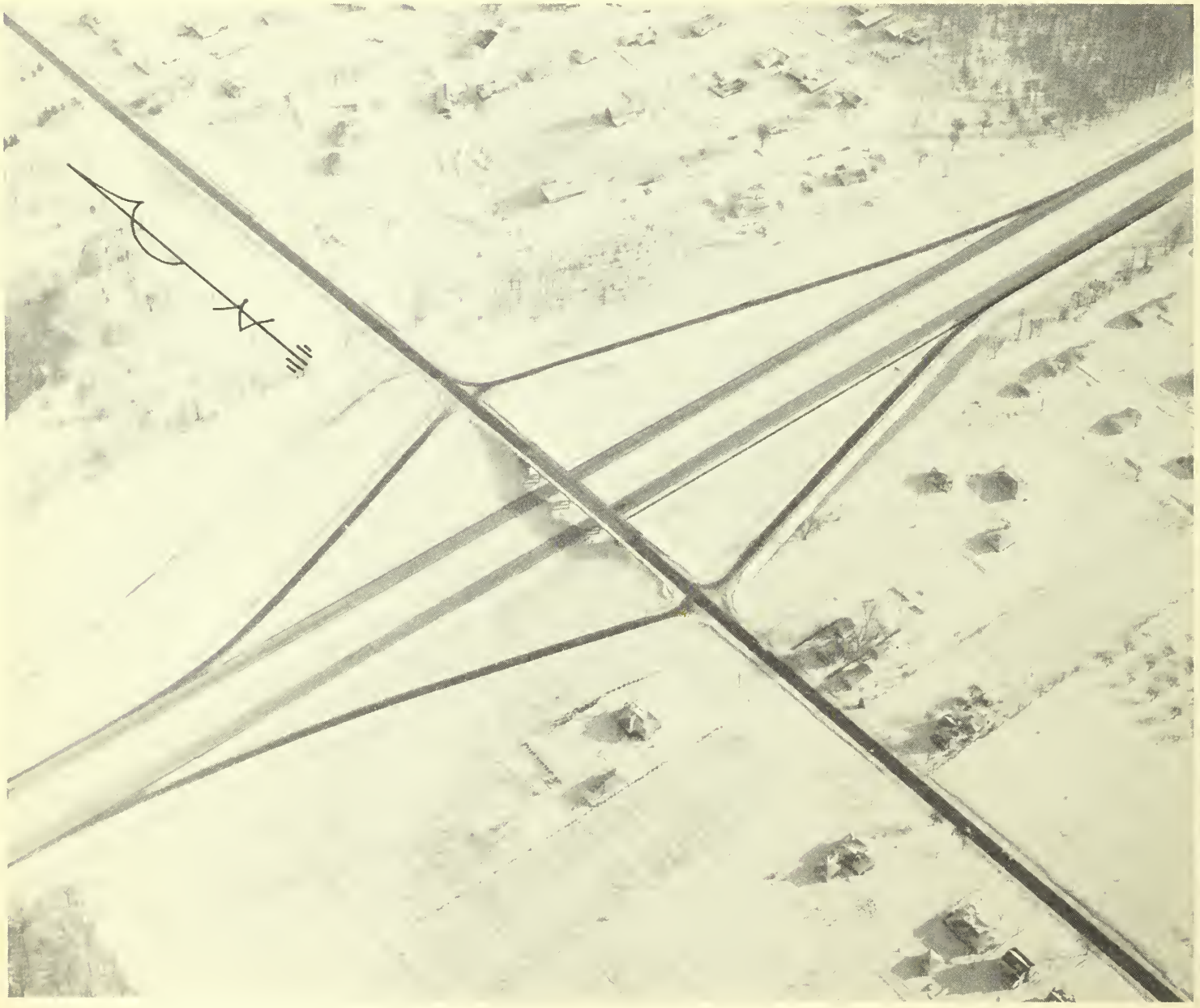

Figure 3.—Residential lots oriented to a modern highway, illustrated by Interstate Route 94, south of Kalamazoo, Mich.

But highways often exert a beneficial influence as evidenced by the reclaiming of blighted areas and the prevention of blight by the removal of substandard structures for a highway improvement. Because of the low prices paid for right-of-way property that is in a rundown condition, construction of the Interstate System has been an incentive for urban renewal efforts in a number of cities. It has been estimated that 50 percent of the Boston, Mass., households displaced for the inner beltway, between Memorial Drive and Massachusetts Avenue, would have been removed eventually during urban renewal projects (*7*). In addition to providing an impetus for clearing slum areas, the payment provided for the direct cost of property taken for the highway can assist localities in clearing out these areas. The Federal Reserve Bank of Boston has referred thus to the effect of highway construction on urban development: "A web of new highways across the land is eating out slums . . . and generally changing the faces of our cities."

Highways can also help prevent blight by providing boundaries between differing uses of land. Since modern freeways are ordinarily broad, well-landscaped, and quite permanent, with relatively few cross-overs, they can be made to serve as effective barriers between areas, such as between residential and industrial areas. James Rouse and Wilfred Owen have compared the Waverly and Hampden areas in Baltimore, Md., stating that Hampden has been able to keep its identity and avoid blight because of its surrounding barriers—including highways (*8*). The Gulf Freeway demonstrates an effective use of a highway facility to separate industrial and commercial uses of land from residential areas, as shown in figure 2.

However, geographical separations caused by highways can sometimes be undesirable; for example, the division of an elementary school district. Obviously, coordination of highway planning with land-use planning is required. As Ellis Armstrong has said, "A highway resembles a hallway with the same need for a blueprint to show what is to be on either side—living rooms, office space, bedrooms, etc." (*9*). Enough experience has been gained with residential areas adjacent to modern highways to demonstrate that the two can be compatible, if they are properly coordinated. Figure 3 illustrates a successful way of locating residential areas near a highway, with deep backyards abutting the highway. More than four-fifths of the 1,239 urban places of 10,000 population and over (the number out of 1,749 that have responded to a Public Roads survey) reported the existence of either a comprehensive, a transportation, or an arterial highway plan.

Relocation of Residents

While more detailed information is needed to show what happens to the people displaced from land taken for highway right-of-way, enough has been learned to suggest that certain adjustments may be typical. Investigations in several different locations have indicated that displaced residents often improve their living conditions, either by upgrading houses that are moved to new lots, or by purchasing better quality houses. In fact, there is sometimes such an upgrading in housing that total assessed valuation for people relocating in the same community is greater than the total assessed valuations for all those who were displaced. However, it is easy to overestimate the highway influence when upgrading accompanies relocation; the highway may in some cases merely hasten an upgrading that would have come later without the highway. The possibility also exists that some people improve their housing conditions because more moderately priced housing is unobtainable, though such cases are believed to be few.

Employment

Modern highway transportation affects employment and employees in several important ways. Improved highway transportation has the effect of bringing additional employment opportunities within commuting distance. Highway improvements also commonly result in a saving to employees, in time or money, or both. For employees working in locations served by a modern highway, such as industrial parks, a further benefit of easy and economical parking ordinarily results.

The reliance placed on highways for commuting to work is shown by the results of a recent survey in 20 States—almost 70 percent of all workers commuted to and from work by automobile (*10*). The importance of bringing additional employment opportunities within commuting distance can hardly be overestimated. Increased employment opportunities increase the likelihood that individuals will be able to find suitable employment—jobs that more nearly correspond to their full capabilities. As the Baltimore Regional Planning Council has stated, ". . . an important goal to be considered is the desirability of maintaining a variety of employment opportunities accessible from any given residential area so that a change of job will not necessitate a change of residence away from established social ties."

Travel Time

That workers are traveling increased distances to work on modern highways appears undeniable; work travel is thought of in terms of travel time rather than distance. Of workers using automobiles for the trip to work, over 10 percent have a one-way travel distance of more than 15 miles, and about two-fifths of them travel 25 miles or more. In Detroit, one out of five workmen has been reported to travel more than 10 miles to work. In Atlanta, Ga., 15 percent of the labor force crosses county lines to employment, and these commuters typically travel about 33 miles on public highways. A number of workers have been found who travel as far as 60 miles to work; and around Boston, Mass., commuters on the modern design Route 128 spend about the same amount of time in going approximately 15 miles to work as others spend in going an average of 8.5 miles on other routes (*11*). This apparent advantage of a modern highway system may of course result in part from other causes such as less settlement or congestion in the area served by Route 128. Figure 4 illustrates the effects that a modern highway apparently had on the commuting pattern of one firm's employees.

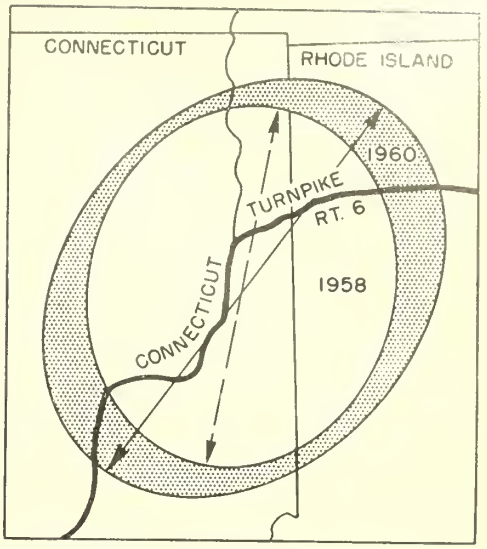

Figure 4.—Extension (1958–60) of labor market for one manufacturing firm that followed construction of the Connecticut Turnpike. Note how this expansion was greatest near the axis of the Turnpike.

Highway Effects on Public Services

Highway effects on public services resemble their effects on many other aspects of American life; some appear to be harmful, others beneficial. In some instances, areas subject to the influence of a modern highway have been associated with increased demands and higher costs for education and other public services, as well as with increased tax rolls to help pay for these increases in public service. Perhaps the most important and beneficial effect of improved highway transportation on public services is that public service facilities are made available more generally and more economically.

Protection services

Protection services such as fire and police are clearly dependent on the availability of efficient highways. The fairly elaborate systems that firefighting organizations employ for mutual aid in case of catastrophic fires provide an example of this important reliance on efficient highway transportation. Police protection can be improved by the highway's effect in making police officers more mobile, by increasing coordination between State and local police, and by permitting use of more centralized headquarters.

Education

Education has undergone several marked changes that appear to be quite directly related to highway improvements. Some of these changes are shown in figure 5. The number of rural one-room schools has declined from about 190,000 to 25,000 during the past 40 years. During this same period, the number of pupils transported in schoolbuses has

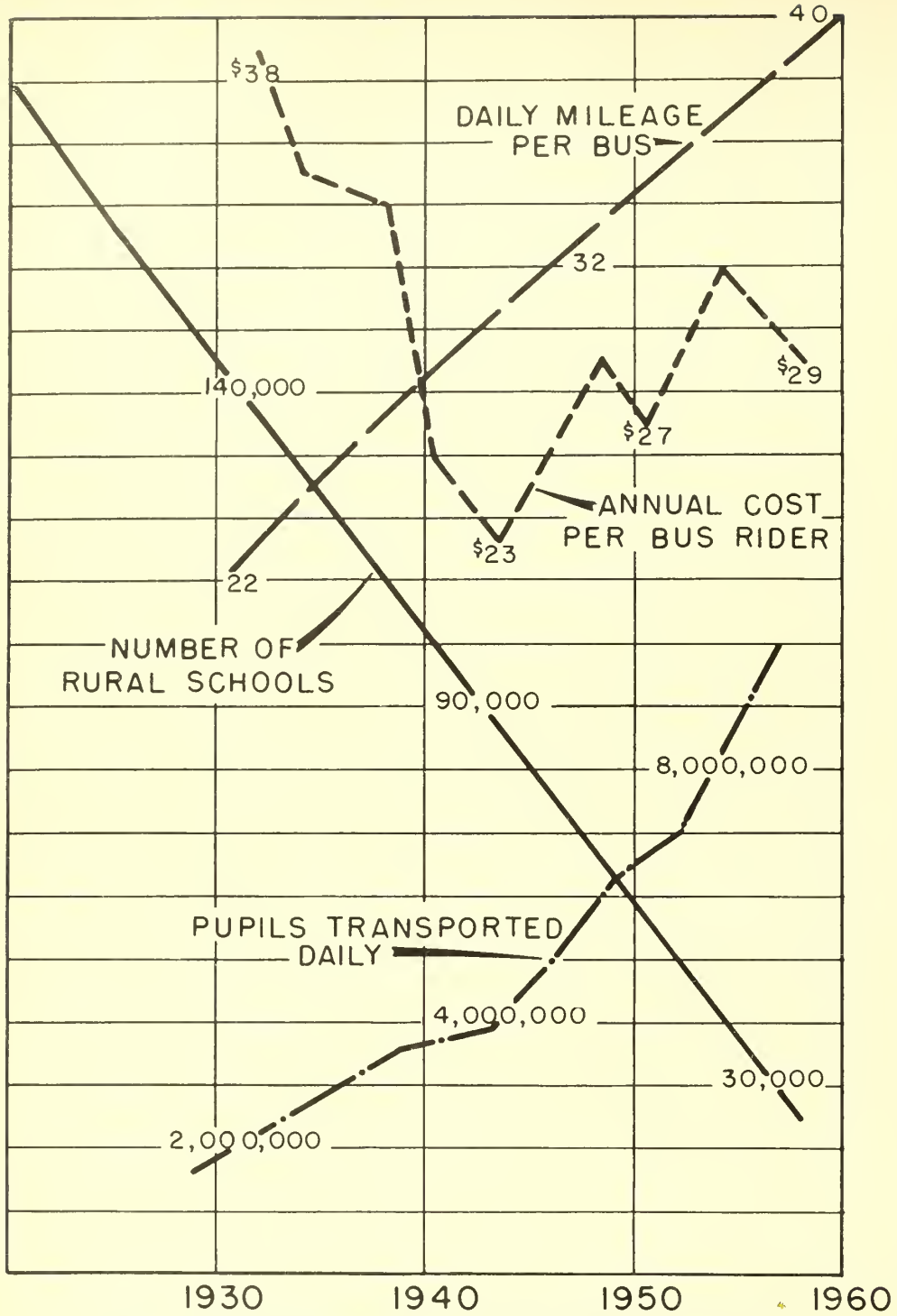

Figure 5.—Relation of new highways to changes affecting rural schools.

increased; and the unit cost of transporting pupils has declined. This lower cost appears to be the result, at least in part, of the improvement of available highways. During the past 30 years, the daily mileage traveled by each schoolbus has nearly doubled—from 22 to 40 miles—and the number of pupils carried by each bus has increased. Other improvements related to education, which appear to be at least partly associated with improved highway transportation, are the gains made in school enrollment—both in absolute numbers and in the proportion of the school age population enrolled—and gains made in the average attendance of enrolled pupils. The absentee rate in public schools in 1900, for example, averaged more than 33 percent of the enrollment compared with an absentee rate of only 12 percent in 1958.

The increasing feasibility of daily highway travel to educational establishments has special significance for higher education. The burgeoning United States population, combined with higher and higher expectations for education, would place an intolerable burden on colleges and universities if these institutions had to provide living facilities for all those attending college. In addition to offering colleges and universities some relief from the necessity for providing dormitory space, commuting from home to college often permits the young people who cannot afford to pay room and board to attend college. Such commuting by auto obviously may aggravate parking problems on campuses and in nearby areas. Figure 6 shows typical changes that have occurred during the past 20 years in the pattern of commuting to an institution of higher learning. The increased volume and greater distances involved are quite noticeable. At many colleges and universities, nearly half of the students arrive by automobile (*12*).

Postal service

Postal service changes also reflect certain highway effects. Improved highway transportation appears to be at least partially responsible for the trend to longer rural delivery routes, fewer post offices, and an increase in the portion of mail being hauled by highway. Changes made in rural mail delivery routes show a remarkable improvement in productivity. The average length of rural mail delivery routes had increased from 27 miles in 1920 to 56 miles by 1959. Table 1 provides additional information about the development of rural delivery routes. In addition to improved highway transportation, the increased distances that rural carriers travel have no doubt depended on the more lenient requirements approved for establishment of rural delivery routes, such as the change from the requirement for four families per mile traveled prior to 1953, to two families in 1959.

Increasing reliance placed on highways through mobility apparently has been a contributing factor in making it possible for the Post Office Department to close a substantial number of smaller post offices without impairing postal service. Thus, from nearly 77,000 post offices in 1900, the number has declined to less than half, approximately 36,000. Many new post offices have been established during this same period to serve areas with increasing populations. While the number of post offices has declined, the volume of mail has increased manyfold; in recent years, the percentage increase in mail volumes has been about twice that of the population.

Health services

Health services also have been affected in important ways by improved mobility made possible because of better highways. This improved mobility obviously benefits many phases of health service—patients, physicians, visiting nurses, health inspectors, health testing units, etc. Medical doctors were among the first users of the automobile. Automobiles permitted them to increase the number of patients they could call on from 5 to 7 a day to, perhaps, 8 to 10 a day. Gradually, however, the practice of doctors to make home visits has changed until now the practice is for patients to visit doctors' offices. The increase in productivity of doctors resulting from this change in mode of operation has been quite spectacular: from 15 to 35 patients now can be seen daily. An American Medical

Table 1.—Rural free delivery routes, 1900–59[1]

Year	Rural free delivery routes			Total annual travel
	Number	Total length	Average length	
		1,000 miles	*1,000 miles*	*1,000 miles*
1900	1,259	29	23	-------
1910	41,079	993	24	303,007
1920	43,445	1,152	27	348,627
1930	43,278	1,335	31	404,738
1940	32,646	1,402	43	424,704
1950	32,619	1,493	46	453,260
1959	31,377	1,753	56	532,677

[1] Source: *Statistical Abstract of the United States, 1959*, U.S. Bureau of the Census, and *Annual Report of the Postmaster General, 1959*, U.S. Post Office Department.

Association study notes that in a rural county in Illinois, 16 physicians in 1950 had provided more service for more people than 42 physicians had provided in 1920.

The effectiveness of such health facilities as hospitals often depends largely on how accessible they are. Travel time to the facility may be all-important. In some instances, as many as 75 percent of all hospital patients arrive by motor vehicle.

Location criteria

Location criteria for public service facilities provide some indication of the effect highways have on public services. In general, service areas are expanding and accessibility is being recognized as a more important factor than that of being near the geographic center of the area served. Thus, police stations may be established on an accessible site with space for parking. Schools may be located away from main thoroughfares to avoid grade-crossing problems. Some public service facilities, such as senior high schools and hospitals whose activities might annoy nearby residents, need locations removed or isolated from residential areas but accessible to arterial highways. Physicians are locating more and more in suburban areas, many of them in medical centers oriented to high-capacity highway facilities. Libraries, especially downtown libraries, need to be accessible to both pedestrian and motor vehicular traffic. Figure 7 shows a dramatic example of the emphasis that has been given to accessibility in locating a library building; this library building in Hartford, Conn., spans the highway.

Highway Influence on Voluntary Associations

The role that improved highway transportation has played in creating opportunities for new associations and in changing many local neighborhood ties is fairly obvious. According to Professor Hawley, highway transportation, ". . . more than any other single factor, has revolutionized the pattern of local relations." With the expanded community made possible with modern highway transportation, ". . . the traditional support of local government, the church, the family, the neighborhood group, is weakened." The services formerly derived from such units, if sought at all, are found in better equipped and more widely scattered units (*13*).

Like many other highway effects, then, it appears that the influence that highways have exerted on participation in voluntary organizations has been both beneficial and harmful in some aspects. The easier participation in church, fraternal, social, civic, or other voluntary organizations, which improved highways permit, is suggested by the results of a Connecticut study. This survey of more than 1,800 suburban families revealed that those families located on hard-surfaced roads had an average per family of nearly two memberships in voluntary organizations compared with an average of only about one-half a membership for those families not located on hard-surfaced roads (*14*).

Churches

Variations in church experience and activity provide some indication of the effect of highway transportation; church activities provide a fairly close reflection of population movements. Thus, suburban churches generally are growing and prospering while churches in downtown areas of large cities and in rural locations are having difficulty in continuing to exist. Much of the general increase in total church membership, now about 113 million, and in the percentage of the population associated with a church, 60 percent now versus 40 percent in 1910, has occurred in suburban areas. A survey of 4,600 Protestant churches, in 23 metropolitan districts and 3 suburban counties, shows that 70 percent of the churches in suburban areas have been growing (*15*). The Catholic church has experienced a similar growth in

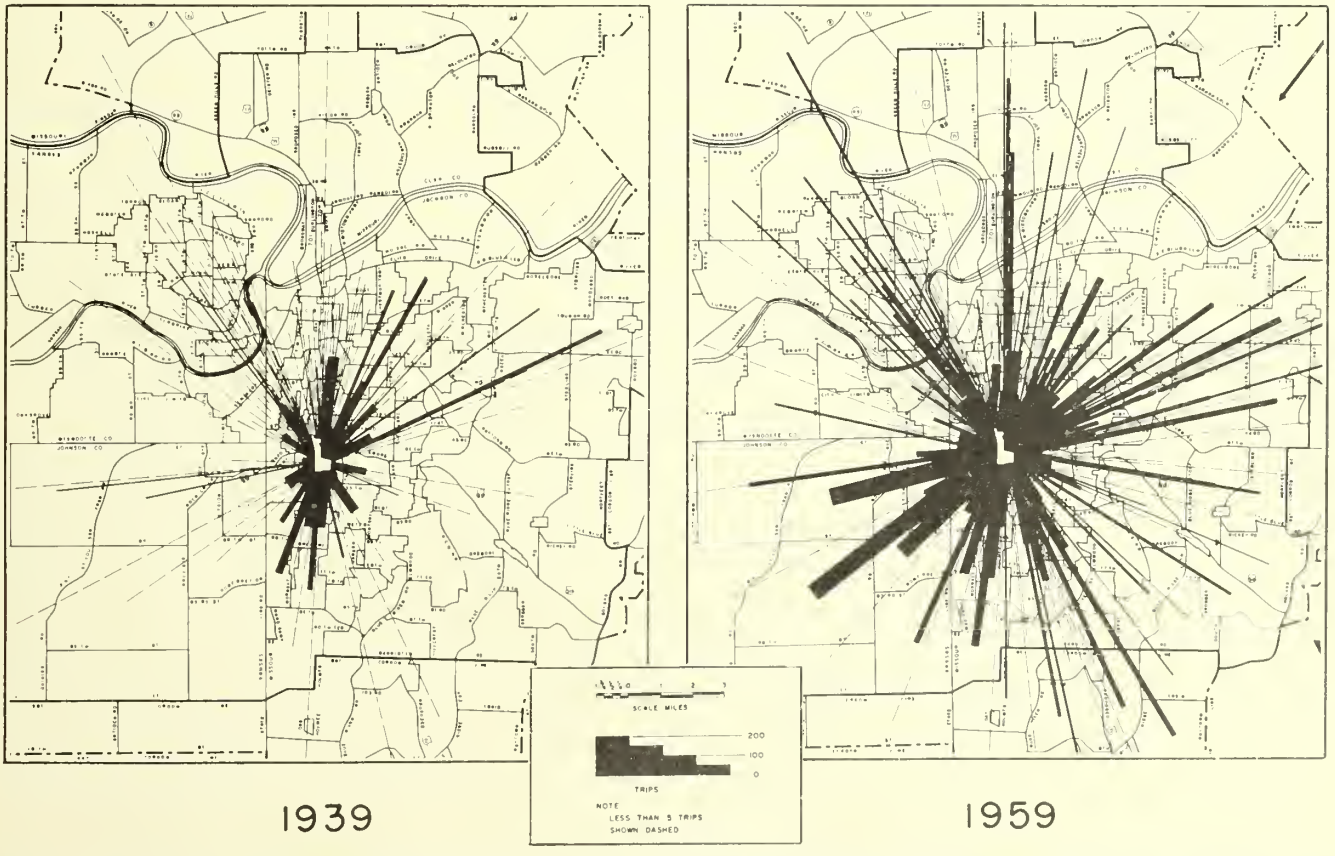

Figure 6.—Changes in commuting pattern for university and college students illustrated by 1939 and 1959 patterns for the University of Kansas, Manhattan, Kans.

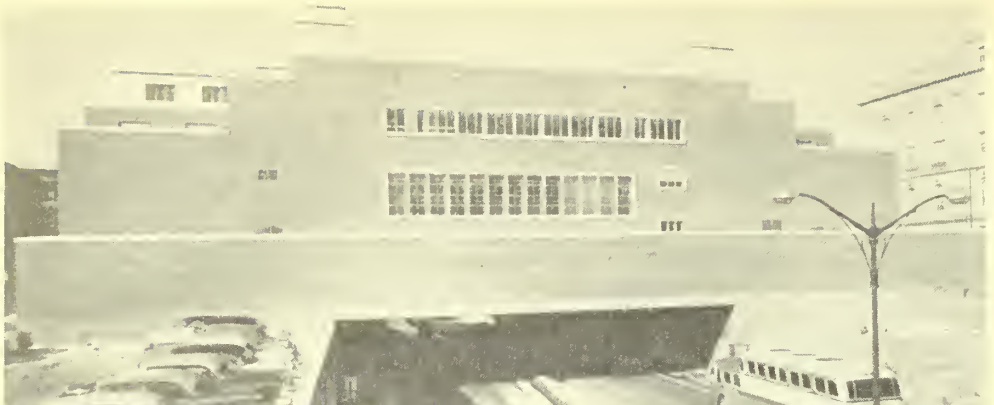

Figure 7.—Public library in Hartford Conn., spans the highway. The windowless section at the base houses the supporting girders and the library's basement bookstack area.

the suburbs; in recent years some 55,000 members have been added each year in the newer sections of Los Angeles alone (*16*). The survey of Protestant churches also revealed that a decline in church activity was experienced by 23 percent of the metropolitan churches, a disproportionate number of which were in downtown locations. In rural areas, Protestant churches closed at the rate of 1,000 per year between 1946 and 1956 (*17*).

Several of these changes in church experience are clearly related to improved highway transportation. It seems reasonable to infer that trips for church attendance are becoming longer. This follows from the fact that the average number of members per congregation has increased from 235 in 1926 to 357 in 1959, even though an increasing portion of those attending church reside in suburban areas, where population may be quite scattered (*18*). In helping to make it feasible for people to congregate in larger groups, highways no doubt have been at least partially responsible for the closing of many rural churches. Rural church difficulties may come about because former members travel farther to attend larger churches and/or because of the " . . . intensified competition for the time of nominal adherents . . ." (*13*). The beneficial effects that highways have in making pastors more mobile for calls on parishioners and, in many cases, for dual preaching assignments seems especially important in view of the growing shortage of clergymen.

The wisdom of locating suburban churches near highways of modern design seems undeniable. The advantage of going on display along a heavily traveled highway may be even greater for churches than for other establishments. Unlike school, work, shopping, and family associations, church affiliation is sometimes so tenuous that many people new to an area will make little effort to locate a church to attend. In the shift of population from rural and central city areas to the suburbs, many people delay affiliating with a church in the new area. In one suburban area in Connecticut, 25 percent of the new residents were found to have no church affiliation; 13 percent of the residents who had been in the community for some time were affiliated with churches (*14*). The need for having their location known to our mobile population—perhaps by locating along a modern highway—is further suggested by the fact that 50 percent of all Protestant church members have joined their congregations during the last 10 years (*19*).

Rural Effects

Improved transportation through highways and motor vehicles has generally been regarded as one of the more important forces underlying the American agricultural revolution (*20*). Both passenger automobiles and trucks are particularly well suited to the needs of farmers and farm operations. Nearly one-fourth of all motortrucks are in farm use, although the farm population constitutes only a small portion of the total population—slightly over one-tenth. And since 1940, the number of farm trucks has been tripled although the farm population has declined. As can be seen from figure 8, both the number of farm operators working off the farm and the number of hired farm workers have been increasing significantly during the same period that the total number of farm operators has been declining. These changes obviously have resulted from the increasing mobility of the farm population; highways are generally the sole means of transportation in rural areas. Farm labor—both migratory and local day workers—arrives practically 100 percent by highway. The importance that highway networks have on the availability of an adequate supply of farm labor is suggested by the fact that certain areas in New Mexico and Nevada, located some distance from major highways, have a chronic shortage of farm labor.

Off-farm employment

The tendency for more and more farm people to work in off-farm employment appears to have been the result of several influences. As a result of the current technological revolution in agriculture, of which highway transportation improvements are an important part, farm people now have more time available for nonfarm employment. Also, the increasing dispersal of industry into rural areas has made more employment opportunities available. From a recent study of rural industry in North Carolina, it was found that one-third of the population in rural areas consists of nonfarmers and that a primary contributor to this has been the automobile. The association of modern highways with increased off-farm employment has been noted in a number of other locations; for example, in Iowa the improved access provided by an Interstate route was credited in part for a higher-than-average rate of off-farm employment (*21*).

Living conditions

Highways are having a profound influence on living conditions in rural areas. Changes in rural life patterns, attributable in part to improved highway transportation, include wider opportunities for nonwork activities, increased opportunities for off-farm employment, and a general improvement in or upgrading of rural areas as places in which to live. That opportunities for such nonwork associations as shopping, recreation, church, lodge, and farm meetings are increased with improved transportation is suggested by the increasing participation in these activities by farm people. In 1920, for example, 12 million attendances at farm meetings were recorded; in 1957, 77 million attendances were recorded, even though the total number of farmers has been declining (*22*). The influence of improved highways in facilitating attendance at farm meetings has been noted especially in Missouri, where highways have been credited with speeding the work of county agents and facilitating attendance at social gatherings and church (*23*).

In providing alternative opportunities for the close family relationships that have been traditional in rural areas, improved highway transportation may tend to weaken such relationships. While the many changes that have altered farm life at such a rapid pace have created problems in family relationships and social orientation, these changes have nevertheless, opened up new horizons and new opportunities for self improvement and enjoyment of life in rural areas.

That the attractiveness of a rural area as a place to live should be enhanced as a result of improved highway service seems only natural, and this development has been noted in several instances. In Montana, for example, a road improvement was rated by 7 percent of the farm people queried as making

the farm home a more pleasant place to live. Among the specific advantages referred to by the Montana farmers contacted were: (1) ease in visiting other farms, (2) elimination of dust, (3) easier marketing and buying, (4) improvement in schoolbus roads, (5) easier commuting to college, and (6) faster service in emergencies, such as medical visits. In Texas, several improvements have been observed in the rural community served by the Camp Creek Road. Following the upgrading of this Road, the number of dwellings along the highway increased and, apparently, reversed the tendency for this farm neighborhood to disintegrate as a result of farmers or retired farmers moving to town. Several of the new house owners said that they would not have built along this highway had it not been improved.

Farms partially taken for highway right-of-way ordinarily experience a sharp impact. As suggested earlier, there are some indications that partial takings have the effect of hastening changes that would occur later in the absence of the highway. Changes, which have been tentatively associated with partially taken farms, include increased employment off the farm, increased farm consolidations, dispersed farm operating units, fewer buildings per farm, and higher farm mortality. It is interesting to note that, with the regrouping following highway acquisition, the farm units losing land for highway right-of-way have, in several instances, ended up with more land.

Highway Effects on Recreation

The different ways in which highways have affected recreation have been commented on recently by Wilfred Owen, who deplores the taking of park and recreation land for highway right-of-way; but he also points out that highway transportation often makes possible the fullest enjoyment of park areas. The acquisition of highway right-of-way has in some cases necessitated the taking of portions of recreational areas; this effect of a highway should obviously be avoided if possible. Criticism has also been leveled at modern highways for the alleged lack of provision of "intellectual stimulation" and "emotional content" (*24*); but highways have often been given credit, along with increased leisure and income, for the tremendous increase in recreational facilities and activities. At a 1958 New York-New Jersey Regional Recreational Conference, for example, the tremendous expansion of the arterial network crisscrossing the metropolitan region was referred to as making possible the phenomenal growth of the park system. In the New York-New Jersey region, the arterial system not only provides easy access to the State park systems but has, in addition, provided the neighborhoods in the city through which it passes with hundreds of acres of urgently needed neighborhood recreation space.

There is, of course, a strong association between highway travel and recreation. In California, the total passenger-miles traveled for recreation has been reported to be as great as for all other purposes, and the strong reliance of many recreational areas on good highway transportation has been well documented. In Yellowstone National Park the number of annual visitors increased from 52,000 before 1917, when no cars were permitted inside the park, to 250,000 annually in the 1917–27 period and to 1.4 million in 1959 (*25*).

Drive-ins and Services for Mobile Public

The various drive-ins and services available for the mobile public are among the most obvious of the ways in which highway transportation is influencing the American way of living. The variety of commercial enterprises or services that have been established as drive-ins or as facilities for motorists is quite remarkable, regardless of whether these highway-oriented establishments are considered desirable. In fact, someone has suggested that a motorist can drive in for almost any service except parking.

Drive-in facilities have been used for such endeavors as banks, churches, laundries, telephones, insurance claim windows, drug prescription windows, tax offices, car washes, mail and library deposit boxes, and motor-vehicle registration windows; and the number has grown phenomenally. The original drive-ins—filling stations—have increased in number to more than 182,000. Other types of drive-in facilities that have become fairly numerous include motels, 58,000; bank windows, 6,000; and drive-in restaurants, 30,000. Drive-in restaurants are among the fastest growing types of drive-in establishments. Three out of each five new restaurants being established are drive-ins, and about one-third of all the money spent at restaurants is spent at drive-ins. There may be some cause for concern about the rapid growth of these eating establishments since objectionable noise and litter problems sometimes are associated with such businesses. An analysis of the relative desirability of retail trade locations shows

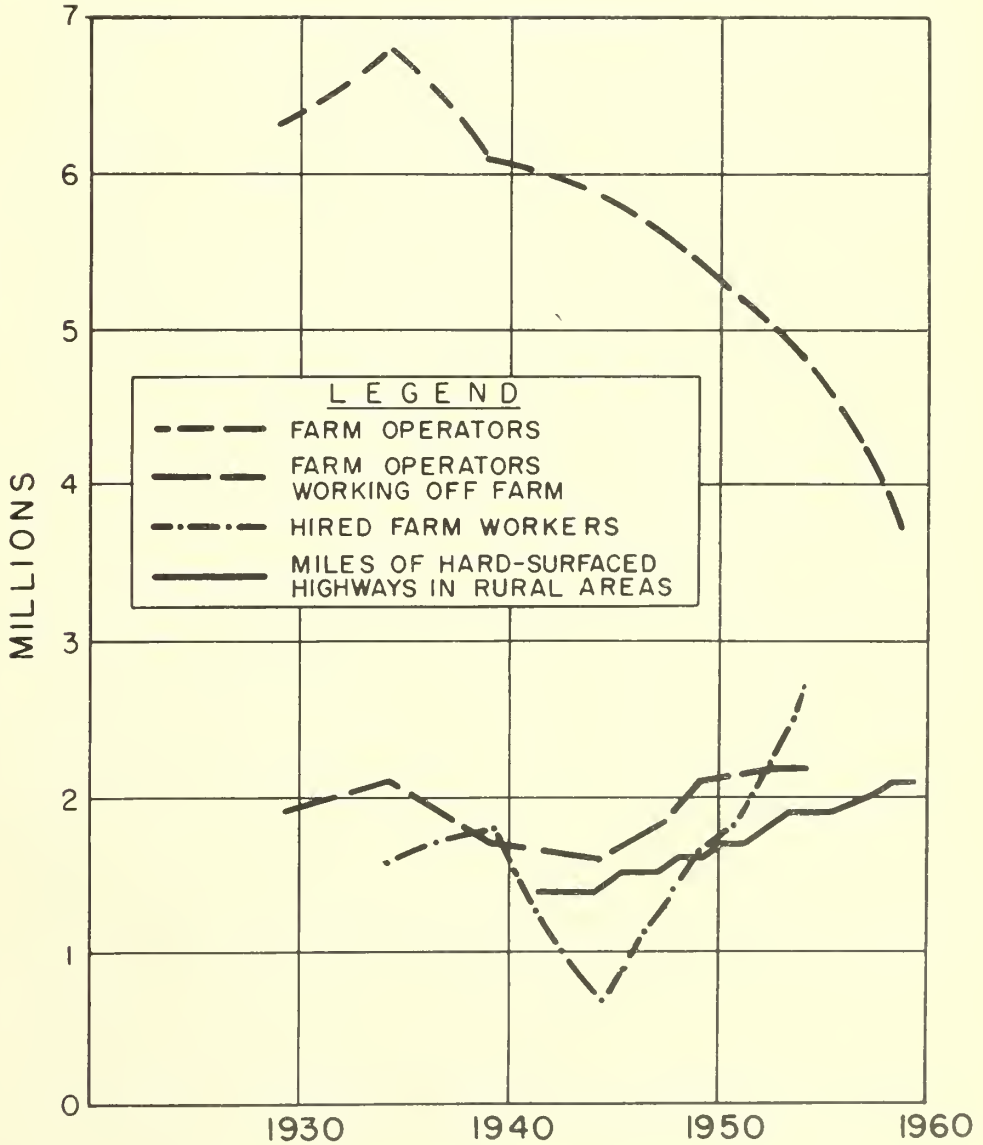

Figure 8.—Effect of highways on farm labor.

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that drive-in restaurants are incompatible with nearly all other types of businesses (*26*).

The growth and variety of other services for the mobile public has paralleled that of the drive-ins. A phenomenal growth has occurred in the number of mobile homes; retail sales of mobile homes have increased from about $13 million in 1930, to $39 million in 1954, to $600 million in 1957. About 3½ million people are now housed in mobile units, the majority in some 13,000 mobile-home parks. The popularity and community acceptance of mobile-home living varies considerably from region to region. As might be expected, it appears to depend in an important way on climate; mobile homes generally have been more popular in southern and western regions of the country. There appears to be a growing awareness of the need for planning sites for these abodes to fit in with residential areas of the conventional type and with other land uses.

Other, but less common, activities associated with mobility on the highways include swimming pools and, curiously, bee hives. A few resourceful beekeepers have been able to stretch out the honey-making season by mounting bee hives on trucks and transporting them between Minnesota and Texas, between southern California and Idaho, and between New York and Florida (*27*).

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New Publications

Highway Progress, 1961

Highway Progress, 1961, the annual report of the Bureau of Public Roads for the fiscal year 1961, is available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at 35 cents a copy.

In this report, the Bureau of Public Roads, U.S. Department of Commerce, presents a review of its accomplishments in connection with the Federal-aid highway program and many other activities. An illustrated publication of 108 pages, this report is a descriptive account of the tremendous progress made during fiscal year 1961 on construction of the National System of Interstate and Defense Highways and on improvement of primary highways, secondary roads, and urban arterials under the regular Federal-aid program. Also included is a description of the highway construction work undertaken directly by the Bureau of Public Roads in national forests and parks and on other Federal lands, and a description of the activities of Public Roads in providing technical assistance to foreign countries to further their programs of highway development.

A description of the significant new legislation contained in the Federal-Aid Highway Act of 1961 and of two important reports submitted to the Congress are included also: the reports are on the new estimate of the cost of completing the Interstate System, and the highway cost allocation study.

Other material in the annual report includes information on the activities and accomplishments of Public Roads in highway planning and design, and urban transportation planning, and on its extensive and varied research program. Included as an appendix in the report are 21 statistical tables covering the progress and activities of the Federal-aid program during the fiscal year 1961.

Highway Statistics, 1960

The Bureau of Public Roads has recently published *Highway Statistics, 1960*, the sixteenth in this annual series. The new bulletin presents the 1960 statistical and analytical tables of general interest on motor fuel, motor vehicles, highway-user taxation, State and local highway finance, highway mileage, and Federal-aid for highways. The 206-page bulletin may be purchased from the Superintendent of Documents, Government Printing Office, Washington 25, D.C., at $1.25 per copy. Earlier annual issues of the series, and a summary of Highway Statistics to 1955, are available from the Superintendent of Documents as indicated on the inside back cover of Public Roads.

Estimated Travel by Motor Vehicles in 1960

BY THE HIGHWAY PLANNING DIVISION
BUREAU OF PUBLIC ROADS

Reported by THEODORE S. DICKERSON,
Highway Research Engineer

MOTOR-VEHICLE travel in the United States in 1960 totaled 718.8 billion vehicle-miles, an increase of 3 percent over the travel in 1959.[1] The travel data were compiled from information supplied by the State highway departments and toll authorities. Total travel for 1961 is estimated at 733 billion vehicle-miles, a 2-percent increase over 1960, based on information for the first 10 months of the year.

The proportions of travel by road system and by vehicle type changed little from 1959 to 1960. Of the 1960 travel, 40 percent was on main rural roads, which comprise 14 percent of the Nation's total of 3.5 million miles of roads and streets. Another 46 percent of the travel was on urban streets, which comprise only 11 percent of the total mileage. Local rural roads, which make up 75 percent of the total mileage, carried only 14 percent of the travel.

Passenger cars represented 84 percent of the vehicles and accounted for 82 percent of the travel in 1960; trucks and truck combinations represented 16 percent of the vehicles and accounted for 17 percent of the travel; buses accounted for less than 1 percent of both the vehicles and the travel.

Average vehicle performance in 1960 differed very little from that reported in 1959. The average motor vehicle traveled 9,652 miles in 1960, almost half of it in cities, and consumed 777 gallons of fuel at a rate of 12.42 miles per gallon. The average passenger car traveled 9,446 miles and consumed 661 gallons of fuel, at a rate of 14.28 miles per gallon. A slight decrease from 1959 appeared in average annual mileage traveled and fuel consumed by commercial buses, while trucks showed a modest increase in both average annual miles traveled and fuel consumed.

The travel and related information for 1960 is shown in table 1 by road system and vehicle type.

Table 1.—Estimated motor-vehicle travel in the United States and related data for calendar year 1960 [1]

Vehicle type	Motor-vehicle travel					Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural road travel	Local rural road travel	Total rural travel	Urban travel	Total travel			Total	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles/gal.*
Passenger cars [2]	225,755	77,528	303,283	284,800	588,083	62,258	9,446	41,169	661	14.28
Buses:										
Commercial	869	154	1,023	1,849	2,872	76	37,789	618	8,132	4.65
School and nonrevenue	597	635	1,232	249	1,481	196	7,556	209	1,066	7.09
All buses	1,466	789	2,255	2,098	4,353	272	16,004	827	3,040	5.26
All passenger vehicles	227,221	78,317	305,538	286,898	592,436	62,530	9,474	41,996	672	14.11
Trucks and combinations	61,262	20,460	81,722	44,687	126,409	11,945	10,583	15,882	1,330	7.96
All motor vehicles	288,483	98,777	387,260	331,585	718,845	74,475	9,652	57,878	777	12.42

[1] For the 50 States and District of Columbia.
[2] Includes taxis; also motorcycles (575,497 registered).

[1] *Estimated Travel by Motor Vehicles in 1959*, by Alexander French, PUBLIC ROADS, vol. 31, No. 7, Apr. 1961, p. 158.

A Preliminary Evaluation of Color Aerial Photography for Use in Materials Surveys

(*continued from p. 20*)

the spring or summer because of the lower sun angle. These shadows are particularly apparent in photographs taken in high latitudes. The intensity and wavelengths of sunlight also vary with the time of day as well as with the latitude. To obtain proper color balance and minimize shadows, aerial photographs should be taken as close as possible to the peak of the solar altitude.

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Appreciation is expressed to the National Park Service for making this project possible; to D. E. Winsor, Federal Highway Projects Division, Bureau of Public Roads, Washington, D.C., for initiating this investigation; to D. C. Harrington, R. A. Bohman, W. Fitzer, and other personnel of the Materials Laboratory of the Federal Highway Projects Office, Region 9, for help and guidance in conducting this project. The interest and cooperation of the Continental Engineering Co., Denver, Colo., which was essential to the success of this project, is also appreciated.

Advance Route-Turn Markers on City Streets

BY THE TRAFFIC OPERATIONS RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

Reported[1] by LAWRENCE D. POWERS
Highway Research Engineer

The study reported in this article was undertaken to focus attention on the possible need for more adequate signing on highway through routes located in urban areas. While major through highways are often expressways or bypasses near large cities, secondary marked routes are expected to be continued through smaller towns and suburban communities. Although emphasis has been concentrated recently on directional signing and marking of routes on high-speed facilities, it was believed that more adequate traffic guidance should be provided on marked routes over urban streets.

In urban areas numerous intersections, complex traffic patterns, competing signs, and varied backgrounds were thought to be confusing to drivers on a through route, particularly to those unfamiliar with the route and, perhaps, traveling over it for the first time. This study showed that a definite benefit was obtained from the presence of one advance route-turn marker but that a second advance-turn marker seemed to be of little added value.

Additional observations were made by a group of junior engineers concerning the study, its results, and other factors related to the effectiveness of the use of advance-turn markers.

Introduction

MANY CITIES without facilities for by-passing through traffic or carrying it on freeway-type facilities must rely on the urban street system to carry marked routes. Even where major through routes in cities are carried on expressways or bypasses, secondary marked routes probably will continue to be carried through smaller towns and suburban communities. It has come to be recognized that more and more attention needs to be given to the advance warning of changes in the direction of a route and the guidance of traffic; most of the emphasis heretofore has been on traffic guidance for high-speed facilities. The need for adequate advance signing is gaining in recognition as being of equal importance in urbanized areas, where drivers are faced with conditions of numerous intersections, marginal conflicts, complex traffic patterns, and many competing signs and confusing backgrounds.

Consider the following situations: An imaginary turn on a fictitious route, in an imaginary city, is marked by a single directional marker at the intersection.

A driver aware of the impending turn in the route, perhaps because he has driven over it and has become familiar with it, may drive along in confidence, get into the proper lane in advance of the turn, signal for his turn, and generally proceed along the route as though it were not marked at all.

Another driver, who may have driven over the route previously but is not familiar with it, perhaps, may remember that he is in the vicinity of a turn but may not be sure of its exact location. Consequently, he may tend to drive more slowly to avoid overshooting the turn, and he may pay less attention to traffic conditions because he is searching for the turn marker.

Yet another driver, traveling over the route for the first time, must rely on route markers and therefore divide his attention between driving and searching for such markers. When he arrives at the turn and sees the directional marker, he may not have enough time or maneuvering distance to make the turn safely.

Suppose that to assist the driver unfamiliar with a route, a marker is placed in advance of the intersection to warn him that the route is going to turn. He will be informed of the impending turn and have time to decelerate safely and to prepare for the maneuver. However, many distracting backgrounds and competing signs in urban areas may pose a problem. Suppose that the driver misses seeing the advance marker because his attention is diverted by traffic or because the marker is obscured by large vehicles. Perhaps, as insurance, another advance-turn marker should be placed in advance of the first one. In an attempt to determine what justification might be found for a second advance-turn marker, or even for one, a controlled study was made of the effect of the presence and number of advance-turn markers on the ability of drivers to negotiate a route in an urban area.

Conclusions

As a result of this study, it has been concluded that a need exists for at least one advance-turn marker placed ahead of the intersections in urban areas where a route turns. The use of a single directional marker at the intersection does not permit enough time for the drivers' responses, and thereby may create confusion, congestion, and possible traffic hazards.

Little evidence was obtained from this study to establish the general value of a second advance-turn marker. However, the second marker may have some value where alinement of route or large vehicles may obstruct a driver's view, and/or where heavy traffic or other distractions may prevent a driver from seeing only one advance marker. Some slight evidence was obtained from the analysis of the turn signal data to indicate that the use of two advance-turn markers also may be advantageous in areas where a long distance exists between successive turns.

Study Site

The route used for this study of the effectiveness of advance-turn markers was a fictitious route, with many turns, through the downtown area of Washington, D.C. The number of advance-turn markers was varied for each intersection on the test route, which was approximately 4 miles in length and had 12 turns. Intersections for turns were selected according to the following listed criteria: (1) a minimum block length of 400 feet in advance of an intersection; (2) signal control;[2] (3) absence of islands or channelized movements; (4) four legs at right angles; and (5) at least two moving lanes on the intersection approach. Of the 12 turns used in the study, 6 were to the right and 6 were to the left. For each direction of turn, three intersections were a short distance from the preceding turn, one to three blocks, and the other three were a longer distance away, more than three blocks. See figure 1.

[1] Presented at the 41st annual meeting of the Highway Research Board, Washington, D.C., January 1962.

[2] It would have been more desirable to have had all the intersections without signals; however, this was impossible in the downtown area. Therefore, to have all intersections as much alike as possible for this study, all those used had signal controls.

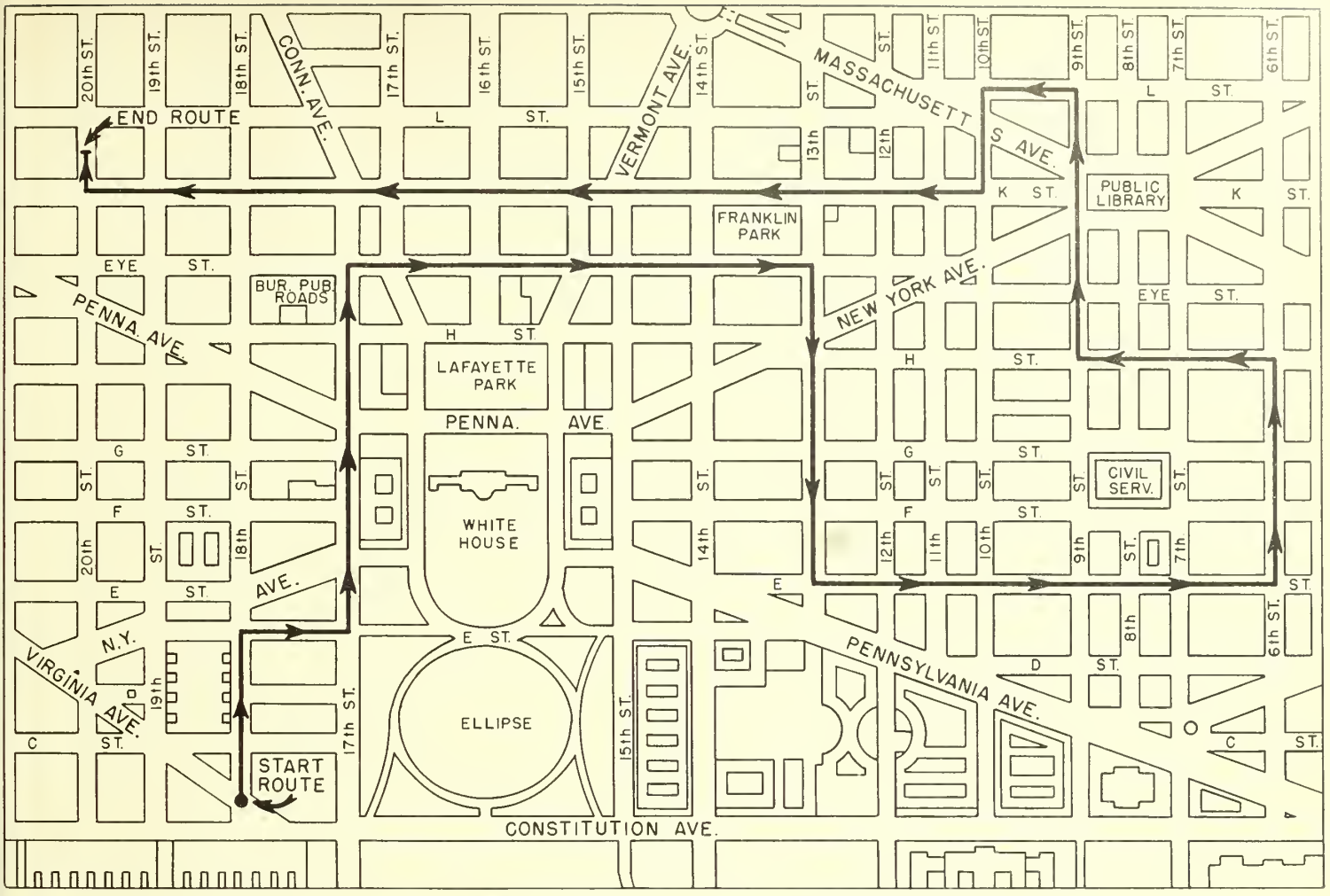

Figure 1.—Map of area in Washington, D.C., showing route for test with advance-turn markers.

A directional assembly with a horizontal arrow was mounted on the far right corner of each intersection where the route turned. The design of the route marker was chosen to conform to the standards in the *Manual on Uniform Traffic Control Devices for Streets and Highways, 1961*. The route marker consisted of a 16-inch white circle on an 18-inch black square (fig. 2), with the designation 00 in 9-inch Series C numerals centered in the circle.[3] Appropriate marker assemblies were made up of this route marker and an appropriate arrow. The arrow plates were 13 inches by 10 inches, the standard size included in the previous issue of the *Manual*. White, nonreflectorized cardboard was used for these markers.

The number of advance-turn markers—none, one, or two—was varied at each intersection on each of the three test days. A balanced design was selected so that each test driver saw each marker combination twice for each direction of turn. The number of long and short turns unfortunately was not balanced with the marker combinations seen by each driver.

All markers were placed on the right-hand side of the street with a distance of approximately 150 feet between all markers for any one intersection. For ease of mounting, the markers were placed on light poles, signal poles, existing signposts, trees, or portable stanchions. The height of the signs varied from 7 to 11 feet from the sidewalk. The higher limit was sometimes used to avoid other signs, signals, parked and moving vehicles, and other obstructions.

Figure 2.—Advance-turn marker assembly used for the study.

Procedure

The 53 test drivers used in this study were all graduate engineers who had been working for the Bureau of Public Roads for from 6 months to 3 years. The drivers were divided into three groups—two with 18, and one with 17—and each group was used for one day. To avoid any emphasis on the route or the markers, the test drivers were told that the aim of the study was to observe driver characteristics in a downtown environment, and that all 53 of them would drive the same course so that a consistent basis for comparison would be provided.

The drivers were told that the observer in the car would not give directions and that they were to follow the marked-out course. They were also instructed to drive in a normal manner and to obey all traffic laws, such as signaling for turns, turning from the proper lane, and obeying speed limits.

[3] The markers were prepared by the District of Columbia Department of Highways and Traffic.

Test runs were made between the hours of noon and 3 p.m., and each subject drove the course once. To prevent the test drivers from overtaking each other, they were started at 5-minute intervals. An observer in each car recorded information as follows: Estimates in car-lengths of the distance from each turn at which the driver entered the proper turning lane and at which the proper turn signal was given, any errors made, and any additional information deemed necessary by the observer. For the purpose of this study, missed turns and correct turns that created hazardous conditions were considered to be errors. Any distances estimated as being longer than 15 car-lengths, approximately 300 feet, were recorded as 15 car-lengths. When a driver missed a turn, he was directed to the point on the route at which he would have been had he made the proper turn, from which point he continued on the test route.

Effectiveness of Advance-Turn Markers

Errors

The desired situation is one in which the driver makes the correct turn without creating a hazard. Therefore, errors, consisting of missed turns or correct turns that created hazardous conditions during this test, were considered one measure of the effectiveness of the advance-turn markers. Without advance-turn markers and with only a directional marker at the intersection, the test drivers made 19 errors, 9 percent of the possible 212 turns for this condition, as shown in table 1. For all practical purposes, the drivers made no errors when advance-turn markers were used. One error was made when two advance-turn markers were in place; however, this error was attributed to extraneous factors.

Table 1.—Errors in following route

Number of advance-turn markers	Possible number of turns	Errors	
		Number	*Percent*
0	212	19	9.0
1	212	0	0.0
2	212	1	0.4

Table 2.—Average distance from intersection at which subject was in proper lane for turn

	Average distance from intersection per number of advance-turn markers—		
	0	1	2
Type of turn:	*Car-lengths*	*Car-lengths*	*Car-lengths*
Short left	5.6	7.4	9.1
Short right	7.0	9.0	10.0
Long left	6.6	11.9	11.7
Long right	6.5	8.6	8.4
All left	6.1	9.6	10.4
All right	6.8	8.8	9.1
All short	6.3	8.2	9.5
All long	6.6	10.2	10.0
Average for all turns	6.4	9.1	9.7

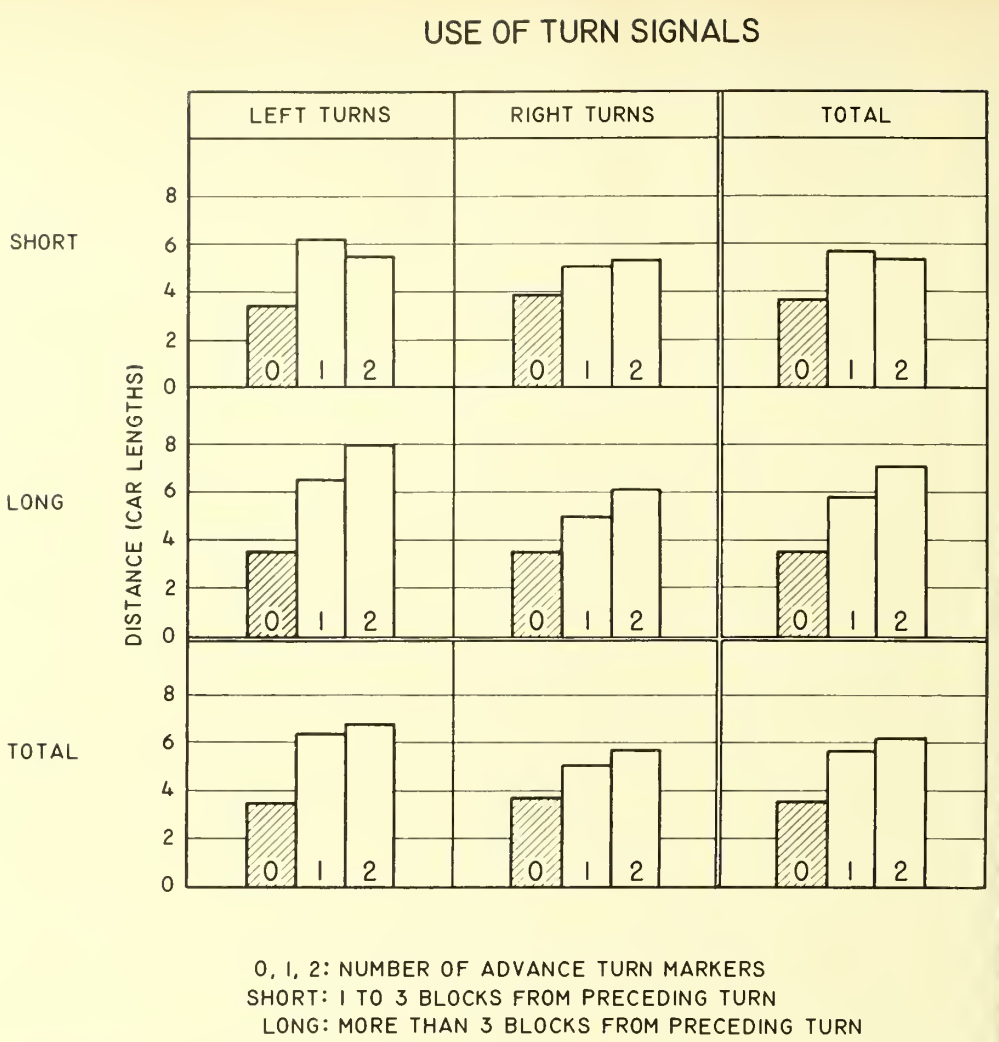

Figure 3.—Chart showing effects of advance route-turn markers on use of turn signals.

A chi-square test performed on the frequencies of errors for this study indicated that there was less than one chance in a thousand that such a disproportionate number of errors could have been made by chance.

Fourteen of the 19 errors were made at left turns; 9 of these 14 were made at the same intersection, where a combination of topography and traffic density were deemed responsible for the high number of errors. As a total of 18 drivers encountered this intersection when there were no advance-turn markers, these 9 errors represent a 50-percent error occurrence.

Table 3.—Summary and analysis of variance for variables indicated

Source	Degrees of freedom	Distance in proper lane			Distance at which turn signal given		
		Sum of squares	Mean square	F [1]	Sum of squares	Mean square	F [1]
Number of advance-turn markers	2	1,302	651	[2] 32.6	833	417	[2] 37.9
Direction of turn	1	32	32	1.6	78	78	[2] 7.1
Subjects	52	2,449	47	[2] 2.4	3,963	76	[3] 6.9
Number of advance-turn markers by direction of turn	2	106	53	2.7	75	38	[3] 3.5
Number of advance-turn markers by subjects	104	1,666	16	0.8	1,073	10	0.9
Direction of turn by subjects	52	925	18	0.9	406	8	0.7
Number of advance-turn markers by direction of turn by subjects	104	1,742	17	0.9	790	8	0.7
Error variance	318	6,331	20	------	3,466	11	------
Total variance	635	14,553	------	------	10,684	------	------

[1] F represents the ratio of the mean squares.
[2] Significant at 0.01 level.
[3] Significant at 0.05 level.

Proper lane and use of turn signals

The distance from the turn at which the driver entered the proper lane to make the turn was considered one measure of the effectiveness of the advance-turn markers. A third measure of effectiveness, and also of the driver's awareness of the presence and direction of the turn, was assumed to be the distance from the turn at which he displayed the proper turn signal. These distances were estimated by the observer to the nearest car-length. All subsequent data analysis was carried out in terms of car-lengths.

For convenience in the field, distances greater than 15 car-lengths were recorded as 15. For distances in the proper lane, values of 15 averaged 11 percent with no advance markers, 25 percent with one advance marker, and 33 percent with two advance markers. For turn signals, the respective figures were 7 percent, 8 percent, and 9 percent. Because many of the distances recorded as 15 may actually represent greater distances, and because they represent a substantial percentage of the field entries for the lane data, the average distances and the variability of the distribution of lane distances are probably higher than were computed. The data on lane distances related to the number of advance markers are shown in table 2. The data have been analyzed as to whether the turn was to the right or to the left and as to whether the distance from the preceding turn was long or short. The same type of analysis was made for the data on average distance from the intersection at which the proper turn signal was given; these data are shown in figure 3.

In almost every case there appears to have been some benefit from the use of an advance-turn marker. When distance in the proper lane is the criterion, for both right and left turns, some additional benefit may have been derived from two advance markers where the turn was a short distance from the preceding turn; one advance-turn marker seems to have been sufficient for long turns. When turn signal distances are analyzed, however, the situation was just the opposite. For both right and left turns, no additional benefit was seen for short turns whereas there appeared to have been some for long turns.

Analysis of Variance

In order to further isolate the factors influencing the effectiveness of the route marker installations, an analysis of variance was performed on each of the major dependent variables (table 3). For the variable, distance in proper lane, the number of advance-turn markers was a significant factor but the direction of the turn did not seem to have an appreciable effect. A high variability was noted in magnitude of response among subjects. However, all the interaction terms involving subjects were not statistically significant; therefore, it is concluded that the relative response between conditions with different numbers of advance markers was not significantly different from one subject to another. It is believed that the possible higher actual variability resulting from the 15-car-length cutoff would not have affected these results.

Figure 4.—Illustration of how trucks may prevent a driver from seeing an advance route-turn marker.

For the variable, proper turn signal, the number of advance-turn markers was also a significant factor in difference of response. In this case, however, the direction of turn does seem to have made a difference, possibly because drivers are more likely to signal for a left turn, which involves a greater probability of conflict, than for a right turn. This is reflected by the data in figure 3. In addition, since the interaction of the number of advance markers with the direction of the turn leads to statistical significance, it is concluded that certain combinations of the number of advance markers and the direction of the turn result in different effects. As in the case of the lane data, there was a high variability in the magnitude of response among subjects. Again, since all the interaction terms involving subjects were not statistically significant, it is concluded that the relative response to the different numbers of advance-turn markers was not significantly different from one subject to another. Combining the nonsignificant interaction terms with the error term and recomputing the ratios of the mean squares did not affect the results.

Discussion

It might be argued that the test subjects were not typical of the average driver. It is believed that their experience in the field of highway and traffic engineering would tend to result in somewhat higher performance in the study. However, this study was concerned with the relative effects of various numbers of advance-turn markers, and it is believed that such relative effects would be reflected in the performance of this group. Perhaps the observed effects would have been more pronounced with more typical subjects.

The number of errors that occurred was lower than had been expected. The reasons for this small number of errors are subject to conjecture but three factors are believed to have been responsible. One has already been mentioned: the subjects were all graduate engineers and, possibly, more alert and more aware than average drivers. Another factor was related to the target value of the sign; this will be discussed later. Finally, it should be noted that the directional markers at the intersections were always mounted on the far right corner, which is also a location for traffic signals in the District of Columbia. For convenience, these turn markers were often mounted on the signal poles, usually very close to the signal. It is presumed that most, if not all, drivers were looking for the signal and therefore had more of a chance to see the marker. Those stopped for the traffic signal also had much more time to see it. However, the chances for being stopped for a traffic signal were the same for all drivers, regardless of the number of advance markers. Therefore, the fact that almost all the errors were made when there were no advance-turn markers

Figure 5.—An intersection where the test route turned left is just beyond the stanchion in the center of the street. When no advance-turn markers were present, 50 percent of the test drivers missed this turn.

indicates that the presence of an advance-turn marker was beneficial in reducing errors.

As was noted previously, any distance of more than 15 car-lengths was reported as 15. Consequently, the average distances for the condition where one or two advance-turn markers were present were very likely higher than those calculated because many of them were recorded as 15 car-lengths. Other factors that might have affected the test results are: the subjects were not specifically told the purpose of the study, and they did not receive instructions to relate the proper lane and the use of the turn signal to their knowledge of the impending turn. They might have been in the proper lane by chance. Conversely, they might have activated their turn signals when close to the intersection even though the markers had been seen further back. However, it is believed that the net result of these factors tended to minimize the observed differences.

FIELD CREW OBSERVATIONS

The field crew [4] made additional observations concerning the study and its results. At several intersections topography, alinement, or physical obstructions, such as transit buses or trucks, obscured to some extent the driver's view of the turn markers. Illustration of how parked trucks may prevent a driver's seeing one advance route-turn marker is shown in figure 4.

Errors

A large percentage of the missed turns came from one intersection in the heart of the downtown area, shown in figure 5. The block on the approach to the intersection was on a downgrade, whereas the intersection itself and the preceding one were level. Neither the intersection nor the directional marker could be seen until the driver had passed through the preceding intersection. While he was on the downgrade, the directional marker, mounted on the far side of the level intersection, was not in his direct line of sight. Because he was not aware of the impending left turn until he was close to the intersection, the heavy traffic often prevented the driver from getting into the proper turn lane.

Where no advance-turn markers were posted, drivers repeatedly had near-misses because they were positioned in the wrong lane. The driver often held up traffic near or at the intersection in an attempt to position himself in the correct turning lane. This situation was considered by the observer to be an error if the turn created a hazardous condition.

When the density of traffic was low on one-way streets, the driver frequently had no realization that the street was one-way. Consequently, on left turns the driver would often signal and turn from the wrong lane. For purposes of analysis, this was not treated as an error because no actual hazard was created; however, these subjects were not credited with being in the proper lane.

Figure 6.—Illustration of how well the white route marker on a black square shows up against both dark and light backgrounds.

Design of Marker and Arrows

One possible reason for the low number of errors was the high target value of the design used for the route markers. Observation in the field showed that the white circle on black square design, based on the new standards in the *Manual on Uniform Traffic Control Devices for Streets and Highways, 1961*, was visible at a much greater distance than had been expected, and long before the numerals were legible. It easily could be picked out from the array of other signs visible along a street (fig. 6).

For the directional markers at the intersection, it was difficult to determine in which direction the horizontal arrow was pointing except at relatively short distances away from it (fig. 7). The direction in which the advance-turn arrow was pointing could be determined at a somewhat greater distance because the position of the vertical part of the shaft gave a clue to its orientation. It would seem, therefore, that the old 13-inch by 10-inch arrows are inadequate, even for low-speed urban usage.

It was observed that some of the test drivers were not aware of any difference in meaning between the bent advance-turn arrow and the horizontal directional arrow. This was particularly evident in situations where two advance-turn markers were used and, because of a short block, the first marker had necessarily been placed close to the preceding intersection. Some drivers, unaware of the difference in the arrows, became confused and almost turned a block too soon.

Figure 7.—Note the differences in legibility of the route marker, numerals, and arrow when this photograph is viewed close up or at arm's length.

[4] The field crew included a group of men who, during their period of training as Junior Engineers in the Washington office of the Bureau of Public Roads, conducted the fieldwork, performed the preliminary data analysis, and submitted a preliminary report: David M. Ham, Thomas E. Knisely, Gardner M. Rice, George A. Rodes, Albert E. Stone, and John R. Webster.

A Preliminary Evaluation of Color Aerial Photography for Use in Materials Surveys

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by JESSE R. CHAVES,[2]
Highway Research Engineer

Findings from an experimental study on the use of color aerial photography for locating and determining the types of materials available for highway construction are included in this article. As a result of the impetus of an expanded highway construction program, significant problems confront location, design, and materials engineers. The introduction of new techniques and modern methods in highway location and design have made many conventional methods of materials investigation obsolete. Photogrammetry and related procedures have brought a new concept to highway location and design by transferring much of the job of highway location from the field to the engineering office. The opportunity to thoroughly review and generally classify the materials along the staked centerline of the highway, as a field operation, has been curtailed. The lead time for the materials engineer to obtain and test samples has been reduced to a point where the required reports and pertinent laboratory test data sometimes may not reach the design engineer in time for his full use. To relieve this situation, new methods are being sought for preliminary materials investigation.

One answer to the problem appears to be color aerial photography. Through this medium, the materials engineer, like the location and design engineer, has been able to bring the investigation primarily to the engineering office. Considerable progress has been made by the use of conventional black-and-white photographic interpretation. However, the relatively few tonal shades of white to black sometimes are somewhat of a handicap: Photographic interpretation used to locate materials sources, to map geologic materials, or to determine soil and ground conditions is dependent to a large degree on color variations. Thus, use of color aerial photographs will enable the materials engineer to obtain more information and permit him to do his work more efficiently and reliably.

Introduction

ALTHOUGH black-and-white aerial photography has been used in mapping for a number of years by Federal and State government organizations and by private groups, the general use of color aerial photography has been somewhat delayed. This article reports on an evaluation made in an attempt to determine the practicality of the use of color aerial photography for materials surveys, particularly by the Bureau of Public Roads and State highway departments.

Black-and-white aerial photography has been used for a number of years by some State highway departments for materials surveys and inventories, by private organizations and individual consultants for mapping studies and explorations for construction materials, and by the Bureau of Public Roads for materials searches in Alaska and several national forests and parks. In addition, the Bureau has sponsored projects in some States for materials surveys and exploration by aerial photography. Black-and-white aerial photographs for many of these studies may be obtained from the Federal Government; they are generally at a scale of 1:20,000.

The delay in use of color aerial photography for materials surveys has resulted primarily from its high cost, slow film speeds, and the poor quality of color reproduction. However, some U.S. Government agencies have been using color aerial photography: The Geological Survey of the Department of the Interior has used colored aerial transparencies for some geological mapping and mineral exploration studies; the Coast and Geodetic Survey of the Department of Commerce has used them for coastal mapping; the Corps of Engineers, Department of the Army, and other military organizations have used them for military intelligence studies, camouflage detection, and special terrain studies; and the Department of Agriculture has used them for detection of diseased crops and trees. Private commercial organizations and universities have experimented with color aerial photography for specialized uses.

In 1958, the Federal Highway Projects Office of Region 9, the Bureau of Public Roads, contracted to have some experimental color aerial photographs taken in Dinosaur National Park in Colorado and Utah. The results of this experiment were so promising that the possible advantages to be gained from using color aerial photography for soil and materials surveys were considered. Consequently, various other projects for aerial color photography were carried out in Colorado, New Mexico, Utah, and Wyoming.

Continuing the use of color aerial photography, the Bureau of Public Roads in 1961 entered into an agreement with the National Park Service for the Federal Highway Projects Office of Region 9 to conduct a comprehensive inventory of construction materials along the major highway system in Yellowstone National Park. The Park Service provided the funds for this work. In addition to carrying out the inventory, the Bureau of Public Roads made a preliminary evaluation of color photography for use in materials surveys. Reconnaissance mapping of significant geologic units, normally of concern in preliminary highway location and design, and aerial photographic interpretation of special ground conditions were carried out in conjunction with this materials search. In the evaluation, the color aerial photographs previously obtained by Region 9 in Wyoming, Colorado, Utah, and New Mexico were considered.

Conclusions

From the evaluation made in this study, it is concluded that good quality color aerial photography can be obtained and that it is of value in searches for highway construction materials. Although black-and-white aerial photography undoubtedly will continue to be used, it is believed that materials surveys can be completed more efficiently and reliably with color photography. The color transparencies have been found particularly useful for materials surveys as they generally provide information that cannot be seen on black-and-white photographs. The success of this proj-

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[1] Presented at the 41st annual meeting of the Highway Research Board, Washington, D.C., January 1962.

[2] Much of the information for this article was provided by Robert L. Schuster, Associate Professor of Civil Engineering, University of Colorado, and Robert J. Warren, Highway Engineer, Federal Highway Projects Office, Region 9, Denver, Colo., U.S. Bureau of Public Roads.

ect and the experience with the other color photography flights in Region 9, for which 700 linear miles have been photographed in color since 1958, have prompted plans for 250 linear miles of color aerial photography during 1962.

As improvements are made in color film, such as the development of faster emulsions with greater exposure latitude that provide greater contrast, color aerial photography is expected to become more useful to highway engineers for a number of purposes. Although this investigation did not encompass the use of color for engineering soils surveys, the casual observations made concerning the identification of soil differences by photographic interpretation appears to be encouraging. An evaluation of color photography for a broad range of geologic materials and variety of soil forming conditions is needed before color photography is adopted for engineering surveys.

It also is believed that color aerial photography will be considered for use in photogrammetric mapping when the technical difficulties have been overcome in the development of a stable film base to which color emulsions will adhere.

Procurement of Aerial Photographs

Although the Bureau of Public Roads had contracted to have 428 linear miles of color photography taken in Yellowstone National Park in 1961, inclement weather and difficulty encountered in obtaining the required color film prevented completion of the contract; only 300 miles were photographed. It is planned to photograph the remaining mileage during the summer of 1962. The color aerial photographs were taken, at a scale of 1:6,000 (1 inch to 500 feet), in single flight strips along the major road system in the park. In addition, several segments were covered by two or more adjacent flight strips with standard sidelap, to furnish adequate coverage of the area. A number of side flights, mainly along minor rivers, were made where potential sources of materials were suspected. Some oblique photographs were taken to supplement the vertical photographs. A small segment of the photography flown at 1:6,000 was flown also at 1:4,800 (1 inch to 400 feet) to permit a comparison of the photographic detail. Black-and-white photographs (panchromatic) at a scale of 1:6,000 also were taken for segments totaling 100 miles. This permitted a comparison of the two types of photography.

Use of Color Transparencies

The use of color aerial photographs for materials surveys may take considerable time and effort in the office and the field. The time and effort required is related to the training of the personnel interpreting the photographs. The education, training, and experience of the color aerial photographic interpreter is significant. A background in the earth sciences, training and experience in aerial photographic interpretation, and a knowledge of highway construction materials, photogrammetry, and highway engineering are highly desirable requisites for aerial photographic interpreters.

Background orientation

The procedure of using color aerial photographs for a materials survey is similar to that used with black-and-white photographs. For this project, a search and review was made of published geological literature pertinent to the areas to be studied: literature such as geologic maps, bulletins, folios, and reports. Information on known sources of construction materials was obtained from Bureau of Public Roads and Park Service engineers, who were familiar with these areas. Thus, as much background information as possible was obtained before the initial study of the aerial photographs was undertaken. The amount of information available and its value varied for different areas. Rather detailed, recently published geologic maps and reports were available for some areas and only old generalized reconnaissance-type geologic maps were available for others. Background orientation of this nature is considered an essential part of a materials investigation.

Upon completion of the literature review, a rather rapid preliminary examination with a mirror stereoscope was made of the transparencies for the flight strips 4,500 feet wide. From this initial examination, the information obtained from the literature survey was correlated with the aerial photographic patterns. Specific geologic features and landforms were marked on the appropriate plastic envelopes in which the color transparencies had been placed. These features were later investigated in the field. This procedure made it possible to plan and more efficiently perform the field work.

Ground examination

The ground examination consisted of examining rock outcrops, geologic materials in highway cuts, and river-cut banks that had been noted from the aerial photographs. Shallow holes were dug with shovel and mattock; and borings were made with a hand auger to expose materials below the ground surface. Materials brought to the surface as a result of animal digging also were examined. Whenever possible, the field notes were made to include descriptions of soils and rocks, and the approximate depths at which the various types of materials were located. Such ground examination is an essential part of the survey and its importance cannot be overemphasized. Reference numbers for field observations were placed on the envelopes of the appropriate aerial transparency. Wherever possible, a stereoscopic examination of the color transparencies was made in the field with a portable lightbox and pocket-lens stereoscope. Color and black-and-white photographs were also taken from the ground during the field investigation.

Examination of transparencies

After the preliminary phase, the color aerial photographs were examined in greater detail. Units such as granite, glacial till, alluvial fans, and landslides were mapped and their boundaries were outlined on the plastic envelopes of alternate photographs in each flight strip. Appropriate mapping symbols were developed for various types of rock, landforms, and ground conditions. Boundaries or areas that appeared doubtful from the transparency examinations were noted and later checked in the field. The descriptions written for the mapping units were included as a part of the preliminary report written for each segment studied during this project. Potential materials sources were noted and sites for future sampling were marked on the envelopes of the photographs.

To assure coordination of procedure with the policy and planning of the Park Service, the transparencies for each of the indicated sources were stereoscopically examined in the office by a Park Service official before any sampling or field investigation with mechanical excavation equipment was undertaken. All the approved potential sources of material were then explored with a truck- or crawler-mounted backhoe, capable of excavating to a depth of about 12 feet. Representative samples were taken for laboratory testing. A brief description of the materials taken from the test pits was written, and the approximate quantity in each potential source was determined by the field crew. (Although not used in this investigation, electrical resistivity equipment will be used in future explorations to help determine the depths and quantities of material in various deposits.) Final reports on the materials investigations, including the results of laboratory tests, were prepared for each individual highway segment in the park included in the project.

Viewing Equipment

A suitable lighting system is required for proper viewing of color aerial transparencies with a stereoscope. Almost any type of lightbox can be used but a source of balanced light—one that provides a spectrum comparable to that of sunlight—is desirable. A combination of fluorescent and incandescent lamps of about equal wattage will provide a balanced source of light. Some means of varying the intensity of the light is also desirable because the number of color distinctions that the human eye can make varies with the intensity of light; more color distinctions can be made at relatively low levels of illumination. To prevent hot spots and provide an even distribution of light throughout the transparencies, a light diffuser is required. Because excessive heat causes color transparencies to curl and may permanently damage them, ventilation holes are required; and it is desirable to have a small air blower in the lightbox to dissipate the heat created by the lamps.

On this project, a small, portable homemade lightbox was used; it was equipped with fluorescent lights and could be operated on either 115 volts of alternating current or 12 volts of direct current. With this viewer, a lens stereoscope was used to examine transparencies in the field. A larger, less portable lightbox was used in the office for viewing with a mirror stereoscope.

Protection of transparencies

When not in use, the color transparencies were stored in transparent plastic envelopes; annotations were made on these envelopes with a grease pencil. The envelopes containing the transparencies were placed in three-ring notebooks, which provided convenient storage and made them readily available for use in the field.

Discussion

The findings of this study represent the result of only one summer's work in which color aerial photographs were used for locating material sources and for generalized mapping. As more use is made of color aerial photography, undoubtedly more information concerning utilization of such photographs will become available. Since the human eye is capable of distinguishing about 20,000 shades and hues of color, it is not too surprising to have found that color photographs have many advantages over black-and-white photographs for interpretation purposes. The use of black-and-white aerial photographs requires interpretation of specific ground conditions, soils or geologic materials, in terms of photographic tones. The number of tones or shades of gray that can be differentiated is extremely limited; many different types of soils or geologic materials of various colors may be reflected by about the same tonal expression. It should be remembered that neither photographic tones nor color alone are used in the identification of specific materials or in the determination of ground conditions. In some instances, the type of landform, gully, or drainage pattern may be the basis for recognition rather than the photographic tone or color.

Acceptable Color Photography

When color aerial photographs are used for materials surveys, relative rather than absolute colors are of primary importance. Thus, slightly off-color photographs are as usable as those showing exact or nearly exact ground colors. This statement should not be misinterpreted as an endorsement for marginal or poor quality work. Visually, it is only possible to tell whether the color registered on the photograph approximates that on the ground. The quality of the illumination of the transparency also affects the visible color, although the true colors may be registered on the film. If the illumination is deficient in a portion of the visible light spectrum, then the colors in that portion of the spectrum cannot be perceived.

Requirements as to endlap, sidelap, crab, and tilt are not as stringent for acceptable color photographs for materials surveys as those for mapping by photogrammetric methods. In general, acceptable color photography must have good definition of images and even light distribution, must be free of clouds, and must have the proper exposure and color balance. Color photography that has been considerably overexposed or underexposed, discolored in processing, or is off the designated flight lines should be rejected. Overexposed color photographs have a washed-out appearance—many of the colors do not register on the film. Obtaining properly exposed photographs becomes a particularly serious problem in areas with little or no vegetative cover because of the excessive reflection of light and the fact that light meter readings are not always reliable in such circumstances. A greater percentage of the ground colors register on underexposed film than on overexposed; for this reason, slight underexposure is more desirable than overexposure. For the most part, the color photographs for this study were properly exposed, and the colors recorded on the film compared well with those observed on the ground.

Camera and films

The camera used for this project was equipped with a lens having a 6-inch focal length with a maximum aperture of f 5.6. The color transparencies from such a camera are 9 by 9 inches (23 by 23 centimeters). The average flight height above ground for 1:6,000 photography was 3,000 feet.

The color film used for this project had an exposure index of daylight-40. This is a reversal color film of the subtractive type that, when processed, gives a color positive transparency. Emulsion characteristics of this type of film usually vary somewhat from roll to roll; therefore, the exposure index as well as the color balance also varies. Because of this, it was necessary to make trial flights over the terrain for which photography was desired, to determine the proper combination of shutter speed and lens opening that would give optimum exposure and color reproduction for each roll of film. A corrective color-balancing filter had been provided with each roll of film because of these variations in film characteristics. Proper color balance must be obtained to assure relatively accurate color reproduction in the aerial transparency. To obtain this color balance and to minimize shadows, the aerial photographs on this project were taken as near peak solar altitude as possible.

Filters

Haze filters are sometimes used in taking color pictures to prevent the overall bluish hue that is caused by the dispersion of light by dust and water particles suspended in the atmosphere. Haze does not present a serious problem with low-altitude photography, but its effect becomes increasingly apparent as the height above ground of the aerial camera is increased. The magnitude of haze also varies with the time of day, season, and geographic location. Because of the use of haze filters, the need for color balance, and the slow speed of color film, aerial lenses for color photography must have larger apertures than those used for black-and-white photography. High quality lenses having good resolution, little color or spherical distortion, and relatively even light distribution are required for color aerial photography. With the development and use of faster color film, the requirements for lighting and size of lens opening are expected to be reduced.

Processing color film

Requirements for the developing and processing of color film are far more exacting than those for black-and-white film and must be met if acceptable color photographs are to be obtained. The composition of developing and fixing solutions and the developing time must be controlled carefully. The temperature of developing solutions, for example, must be maintained to within $\pm 1°$ F. Considerable care also must be exercised while drying or handling the film. Special, rather costly equipment is needed for adequate processing.

Color film should be developed within 24 hours after its exposure; when this is not possible, the exposed undeveloped film should be refrigerated. Because of the tendency for the dyes in unexposed color film to change with time, particularly in hot, humid climates, all unused color film should be stored at temperatures ranging from 45° to 65° F.

Advantages in Use of Color Photography

One obvious advantage in the use of color aerial photographs is that cultural features such as highways, trails, buildings, and aerial targets are more readily identifiable than on black-and-white photographs. Also, it is easier to orient oneself in the field by use of color aerial photographs because the terrain and culture appear to be more natural. Image definition is better on color aerial transparencies than on black-and-white prints because the use of prints results in some loss of detail.

Wet soils, organic soils, boggy ground, and seepage zones can be easily identified on color photographs because of the green grass and other vegetative growth in these wet areas. The brownish color of the organic soils is readily recognized. Minor drainageways, poorly drained depressions or swales, and seepage areas in landslides show quite clearly. On the black-and-white photographs, these areas have generally darker photographic tones, but identification or delineation is not as positive and, in some instances, the conditions are not apparent.

The identification and delineation of such rock types as granite, rhyolite, basalt, limestone, shale, and sandstone are greatly facilitated by color photography; large boulders in glacial till can be identified more readily. Color is particularly helpful in instances where the rock fracture pattern or other features are not distinctive and cannot be used for identification. In one instance, however, the fracture pattern for a granite was so distinctive that it appeared equally well on both the color and black-and-white photographs.

Various vegetative types can be identified more readily on color than on black-and-white photographs. It is not always possible to correlate a particular type of vegetation with a specific type of material or ground condition, but color photography will be useful wherever reliable correlations can be made. For example, aspen growing on alluvial fans in southern Colorado clearly outlined the extent

of these deposits. Dense timber cover that completely obscures ground detail is a liability in aerial photographic interpretation with both black-and-white and color photographs. Thus, a dense cover of lodgepole pine in one area of Yellowstone National Park prevented the detection of local, thin deposits of glacial sand overlying volcanic flows.

In several cases the colors of certain features were not of the same density in adjacent pictures—vividly colored algae growing in hot springs appeared in almost true color on one photograph but were almost completely washed out in the next. This apparently was the result of the difference in the angle of reflected light from the ground for successive plane positions. Although this change in color density may be obvious when individual photographs are compared, it generally is not apparent when pairs of photographs are viewed stereoscopically. A similar difference in photographic tones also was noted on black-and-white photographs of sand and gravel river bars.

Photographic Scales

Color photographs were taken at several scales so that a comparison could be made as to their relative value for materials surveys. These scales and their respective flight strip widths were: 1:4,800 (1 inch to 400 feet), 3,600 feet; 1:6,000 (1 inch to 500 feet), 4,500 feet; and 1:12,000 (1 inch to 1,000 feet), 9,000 feet. All transpareneies had an image area of 9 by 9 inches square. This study of photographs at different scales confirmed the experience in Region 9, which had indicated that a scale of about 1:6,000 would be the most desirable one for use in locating sources of materials.

Use of a scale at 1:4,800 permits more ground detail to be observed but the flight strip's width is one-fifth less than that for photographs made at the scale of 1:6,000. The advantage gained by having the extra 900 feet in the flight strip's width with the 1:6,000 scale more than offsets any slight advantage gained by use of the larger scale. The additional detail on the photographs taken at the 1:4,800 scale did not help appreciably in the detection of materials sources or in the determination of ground conditions. Photographs taken at a scale of 1:12,000 showed insufficient ground detail for optimum use; this smaller scale, however, did provide greater width of coverage and permitted a broader overall view. Examination of the photographs at this smaller scale required more time for study than those taken at a scale of 1:6,000 and many uncertainties remained regarding actual ground conditions. On the basis of the evaluation of these photographs, it is believed that use of scales ranging from 1:6,000 to 1:8,000 will provide sufficient ground detail and width of coverage for optimum use in materials surveys.

Cost of Color Photography

The cost of color photographs should be considered in relation to actual savings in time, money, and increased information made available through their use—not by the actual cost alone. The added advantages and savings gained through the use of color are not always measurable. The costs involved in procuring either black-and-white or color aerial photographs are small, if not insignificant, compared to the final cost of constructing a highway. If a single, good source of material is located for a project or if the haul distance to a suitable source can be shortened appreciably, the cost of photography and for the personnel involved may well be paid for many times. Similarly, the savings made by recognition and avoidance of unstable ground, hard bedrock, seepage areas, or other poor ground conditions in projecting a preliminary highway alinement are obvious.

The average price ranges per linear mile for 9- by 9-inch color aerial transparencies, shown in the following table, are for photography obtained by negotiated contract in Region 9; prices may not be the same elsewhere. The prices shown here included the cost of the plastic envelopes in which the transparencies were placed.

Scale	*Cost per mile*
1:4,800	$43.75–$60.75
1:6,000	38.25– 52.75
1:12,000	27.00– 33.50

The only available price for black-and-white photographs is for those at a scale of 1:6,000. The cost ranges between $20 and $27.50 per linear mile as compared to the range of from $38.25 to $52.75 for color transparencies at the same scale. The conventional photographs are printed on double-weight paper with a semi-matte finish. Two sets of black-and-white prints and an uncontrolled photographic index are included in this price.

It has been the experience of the Federal Highway Projects Office, Region 9, that color photography may be obtained at somewhat lower prices through competitive bids, but that the quality is often poor and not acceptable. Price quotations will also vary with the number of miles of photography required per job, and the distances involved in getting a plane to the area to be photographed. Color photography in Region 9 generally has been taken at the same time as the black-and-white photographs used for photogrammetric mapping.

Color aerial prints, rather than color aerial transparencies, are not acceptable for use in materials surveys because of their lack of good color reproduction as well as their excessive cost. Relatively large emulsion shrinkage of color prints causes them to curl with small changes in temperature and humidity. The high cost of color prints is a direct reflection of the skill and experience required and the difficulties that may be encountered in obtaining and producing color pictures of acceptable quality. Recently a price of $4 to $5 per print was quoted on color prints when the order was for 50 prints or more.

Care Required for Transparencies

Greater care is required in handling and working with color aerial transparencies than with black-and-white photographic prints. If a transpareney is lost or damaged it cannot be replaced, as with prints. Furthermore, the reproduction of color transparencies is costly and the color reproduction from the originals cannot be relied upon. A disadvantage to the use of color transparencies is the need for suitable illumination to view them in an office and, if they are to be used in the field, a special portable lightbox adapted for battery operation is required. When they are used in the field under extremely hot and dry climatic conditions, the transparencies will curl; but, under similar conditions, black-and-white photographs printed on double weight paper also curl and become brittle.

Procurement of Quality Aerial Photography

Equipment used to process color film is rather expensive and some commercial organizations have neither the equipment nor the experience in taking or processing color photographs that are required to produce a good quality job. For this reason, color photography has been procured in Region 9 by negotiated contract, in order to obtain a high quality product at a reasonable price. Unless a State highway department has had previous experience with a commercial organization, it is recommended that, before a contract is entered into, the organization concerned be required to submit sample photographs taken of the area at the desired scale. Samples at other scales or at the required scale of other areas should not be accepted as evidence that the organization is capable of producing acceptable photographs for a particular area.

The type of camera and lens used for aerial color photography is highly important. There has been a tendency for commercial organizations to use lenses with focal lengths of more than 6 inches, such as 8¼ inches or 12 inches. Although the use of longer focal-length lenses produces better light distribution, which provides more uniform density in the picture, a higher flying height is required to obtain pictures at a given scale and this, in turn, increases the haze effect that causes the undesirable bluish hue in color transparencies. Quality lenses with shorter focal lengths do not produce noticeable vignetting and the haze is less of a problem because of the lower flying heights required. Experience on this and other projects in Region 9 indicates that the quality of a camera for color aerial photography for materials surveys should be equal to or exceed that of a Zeiss RMK 15/2 equipped with a Pleogon lens. No attempt was made to evaluate various types of color aerial film in this study. The contractor used Kodak Ektachrome Aero Film; he had previously produced acceptable color aerial transparencies with this type of film.

The season and time of day are important factors that affect the quality of both color and black-and-white photographs. Preferably, color photographs should be taken during the interval between the complete disappearance of snow and the leafing of deciduous trees or between the time the trees are bare and the first snow. Photographs taken in the fall have longer shadows than those made in

(*continued on p. 11*)

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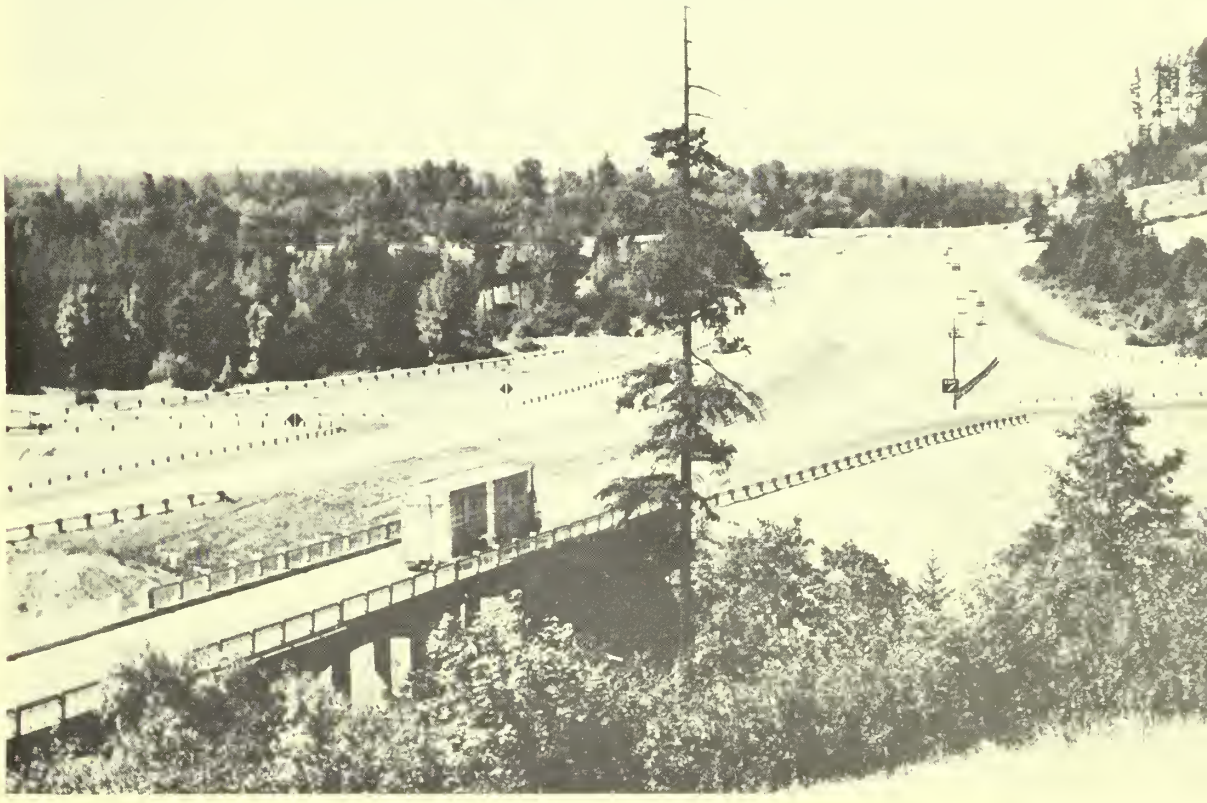

View of the North Jefferson Junction in the North Albany Junction section of the Oregon-Pacific Highway, Interstate Route 5.

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Economic Evidence in Right-of-Way Litigation

BY THE
HIGHWAY AND LAND ADMINISTRATION DIVISION
BUREAU OF PUBLIC ROADS

Reported [1] by SIDNEY GOLDSTEIN,[2] Chief, Economic Impact Research Branch

The information presented in this article is the result of research directed toward providing facts that will be helpful in obtaining legal acceptance of economic research evidence in highway right-of-way litigation. Research evidence has been recognized in other fields of law, and it is believed that such recognition should be extended to highway litigation—economic fact should serve the court in establishing legal fact. A description and analysis of present evidentiary rules and tests are included, and suggestions have been made for the utilization of various types of evidentiary material. Hurdles that must be overcome before some of the evidentiary material becomes fully acceptable also have been described, and some of the means for countering judicial objections have been enumerated.

Acceptance and utilization of economic research evidence, such as that provided by land economic studies, severance damage studies, and economic statistical studies, is considered desirable for calculating just compensation, especially for partial takings for highway construction. Partial takings of property on a wholesale basis is a relatively new item in condemnation law. For example, in connection with the National System of Interstate and Defense Highways, approximately 90 percent of its 41,000 miles will be obtained from rural areas, and acquisition of almost any rural parcel of land will involve a partial taking. Heretofore, valuation by appraisers and realtors has been based primarily upon experience with entire parcels of land. Now, when an evaluation must be made for the remainder after a partial taking, damages must be determined also for the severance, and in some States the benefits accruing to the property from the highway must be segregated from the damages.

The benefits, including savings of both time and money for governmental entities, to be gained by the use of economic research as direct evidence of values are pointed out in this article. It also is proposed that legal counsel and courts re-examine their position with regard to acceptance of currently available economic data and that obtained from systematic research procedures.

ECONOMIC DATA IN CONDEMNATION PROCEEDINGS

Introduction

THE MERGING of ideas from two fields even under ideal circumstances is a difficult rocess. In the case of the disciplines of law nd economics, however, this process has been oing on for many centuries; present commerial legal practice evolved from the need for ules to implement various institutional relaionships in our economic affairs. As a conequence, the development of procedural and ubstantive law involves a recognition of the usiness climate. While the law recognizes arious economic arrangements, newer means f ascertaining facts exist today. The newer echniques of factual presentation can aid ighway and other public officials greatly in heir determination of fair compensation for roperty acquisition. To aid in the refinement of this fact gathering and in an attempt o determine true indicators of value for use n legal proceedings, the Bureau of Public Roads and many State highway departments are engaged in right-of-way and legal research studies. Included in this research are land economic studies, severance damage studies, and economic impact studies.

From such research activities, it is hoped that it will be possible to supply the realities behind the market and the willing buyer-and-seller concepts, abstractions that have been defined fairly specifically in case law, as will be described later. Substantial savings to the Government, as well as verification and justification of condemnation awards, are expected from this research.

The partial takings of property for highway purposes has made more significant the current law of severance damages and the proof necessary in such cases. Although some 9 out of 10 condemnation cases are settled prior to recourse to judicial procedures, those that find their way to the courts ofttimes represent widely varying amounts of valuation for the same parcel of land. The pattern of payments of damages, in present as well as in future cases, is influenced by these interpretations and decisions arrived at in open court. The damages awarded in these cases tend to establish the basis for the level of awards and damages in present appraisal practice since they become part of the jurisprudence kit.

The material presented in this paper on the economic orientation of condemnation cases, and the suggestions for the utilization of various types of evidentiary materials in such cases, indicate a belief that economic fact should serve the court in establishing legal fact. Existing legal practices, with respect to the admission and use of research evidence in courts of law, indicate that economic research techniques have made definite contributions to the judicial fact-finding and decision-making processes. Such findings almost invariably have been confined to areas of commercial litigation with the exception, however, of certain scientific tables and calculations, which are said to be admissible and competent evidence because the demands of custom and practical convenience make them generally, if not universally, acceptable.

Study of research findings

This study of the experience with economic research findings, where they are judicially acceptable, along with the indicated needs emerging from condemnation proceedings for factual data that best can be obtained by research methods, sanctions the admission and use of such findings in land valuation cases. Such sanctions, if adhered to, demand a reshaping of the rules of evidence, which now prohibit entrance of economic research findings as independent evidence in condemnation proceedings. Such a change in evidentiary procedure only summons the next step in the evolutionary process set in motion by the admission of comparable sales of particular parcels. If evidence of sales of comparable parcels can be introduced in piecemeal form, through the drawn-out procedure of separately and individually establishing the collateral issue of comparability followed by evidence of the sales prices, better procedural methods of introducing evidence pertaining to land values should be welcome because of the time factor and the need for better information.

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[1] Presented at the 41st annual meeting, Highway Research Board, Washington, D.C., January 1962.

[2] William H. Stanhagen, formerly Chief, Highway Laws Project, assisted during the planning stage of this article; and law students who were employees of the Bureau of Public Roads, Joseph T. Sweeney and Carrie L. Fair, performed research and contributed to the preliminary preparation of this article.

Land economic studies, severance damage studies, economic statistics, and other research data can provide these better procedural needs and also overcome the shortage of land valuation data, essential in the determination of just compensation. The goal for all condemnation proceedings is the award of just compensation. Because such an award depends on property value, damages, and frequently benefits assigned to the land in question, it follows that these may be determined more accurately by objective research methods. The products of these methods could serve as a means both for substantiating or cross-examining expert testimony and for independent evidence, especially on issues such as: (1) after value, where the before-and-after formula is applied; (2) severance damages; (3) special benefits; and (4) general benefits. Therefore, better valuations of damages to properties, especially to remainder parcels, would be expected.

Filling in Evidentiary Gaps

Participants in the judicial process in eminent domain proceedings for highway purposes have recognized the need for sharpening factual presentations in condemnation cases. A recognition of this need has been seen in the recent emphasis on pretrial practice (*1*),[3] on uniform expert appraisal testimony (*2*), and on severance damage studies (*3*). In this article the major emphasis has been placed on severance damage studies; a brief reference has been made to the other two items.

An objective for this article is to fill the gaps in evidentiary practice so that both the court and the jury will be assisted in their decision-making roles. The public and individual property owners will benefit by any reduction of guesswork that may be inherent in courtroom valuation of property. Concerned by the inadequacies of the factual presentations currently being admitted in the courtroom, various leaders in the right-of-way, appraisal, and legal fields have indicated a need for devices in dealing with certain types of property (*4*).

As an example of this concern, a specialist in right-of-way has referred to the valuation of severance damages in the following manner: "One of the most difficult phases of appraisal work is the assignment of the proper value of severance damages to properties in highway right-of-way acquisition" (*5*).

The desire of right-of-way officials to provide for fair and accurate compensation has been stated, as follows: "It is the general intent that owners of parcels that are severed or reduced in size by right-of-way acquisition be compensated as accurately as possible for damage incurred—that payment be no more, no less, than the true value" (*5*).

Evidence in condemnation cases turns on the question of the value of the property taken, and in partial taking cases on this as well as damages to the remainder. In accordance with the established valuation procedures, properties are analyzed in terms of before and after the taking. In this connection, it is essential to realize that market value must always be an estimate. Even under the most refined expert appraisal, no two properties can ever be exactly alike in value because of the intangibles of location and quality, as well as the differences in background and training of appraisers. Since estimated values are the bases for the determination of fair and just compensation to a property owner, it is worthwhile to ask what can be added to the right-of-way official's technical equipment that also will aid the courts in increasing the accuracy of the estimate of value.

The most usual means of proving value in a court proceeding is by expert testimony in which experts or informed individuals on both sides seek to impress the jury with the defensibility of their valuations. The recognition of new legal evidentiary devices has indicated that the law has progressed substantially. Therefore, the type of evidence generally used in condemnation proceedings may be improved to meet present fact situations in highway condemnation cases. An analysis of the current status of the law of proof and of the admissibility of economic research evidence, such as statistical surveys and samples, and the hearsay objection to such data will be presented later in this article. The particular significance of various types of land economic studies to the highway lawyer and appraiser are discussed in the following paragraphs.

General Economic Impact

In condemnation law, the courts introduce some legal constructs; namely, market value and highest and best use as the determinant of fair and just compensation. Just as in other areas of law, that is in torts, the concepts of the prudent man and the reasonable man are used. Such generalized concepts are needed in order to furnish standards so that a fair decision may be obtained for all concerned. The impact of eminent domain proceedings on the general public has grown with the tremendous building programs currently in progress, some of which are: The Interstate and other Federal-aid highway programs, State and county highway construction efforts, urban renewal and redevelopment, reclamation, flood control, parks, and other programs involving the assignment of compensation.

Despite the technological advances illustrated by all these public works programs, the means of proof has tended to remain within the same paths. Further information is presented in this article in connection with the discussion of pertinent laws of eminent domain and evidence. Courts normally, however, adapt their current procedures to new problems, for this is how growth obtains in the law to handle growth in the economic system.

When an expert witness is giving his opinion of the value of a parcel taken and damages to the remainder, knowledge of land value trends is an indispensable item to him. He would like to be able to ascertain certain general trends in the area so that he can give due credit to such general inflationary or deflationary movements in arriving at his estimate of value. General land value studies provide him with the expert information that he requires.

Information available

It is decidedly difficult and expensive for the individual appraiser to make a thorough study of all land value items in an area. For this reason, the Bureau of Public Roads, State highway departments, and various universities through economic impact studies are making available, to the appraisal profession and highway legal counsel, the types of information needed (*6*). Some of the background of these economic impact studies is given in the following paragraphs. The earlier studies of the 1920's were concerned with rural land values. References to the results of these and many recent land value studies, perhaps 50 in number, have appeared in the report of the highway cost allocation study, prepared for the U.S. Congress by the Bureau of Public Roads (*7*). In all, about 100 highway impact studies have been completed and presently about 40 are underway in some 35 States.

The impact studies have utilized various kinds of approaches. In general, the methodology encompassed what has been called a before-and-after technique—an analysis of some period prior to a highway improvement compared with a period after the completion of the improvement. Wherever possible, geographic areas subject to highway influences were compared with similar areas not subject to highway influence, in order to isolate to some degree the impact of the facility. Although the subject matter in these studies varies widely, the concern with right-of-way is seen in these studies because analyses of land value are usually a component part of any study for evaluating the impact of a highway.

The results of the experience gained in these studies have, in some instances, found their way into the courts, generally through expert presentation. One of the early impact studies that dealt with 2,500 sales in the Houston, Tex., area was used in a Mississippi case as a basis for expert opinion (*8*). Other studies provide findings that could be useful in legal disputes, such as in the Baltimore Beltway study of subdivision property where little or no damage was found from highway proximity.

Even when an appraiser does not have access to such economic impact studies, he impliedly uses similar information in his evaluation of a parcel. His experience and education become the basis for his expert opinion. Nevertheless, it is opinion testimony and, as such, it is subject to all the attacks inherent in the cross-examination of any opinion. Neither side to a legal dispute generally has adequate information to evaluate the trends, for the reasons of time and expense. It is hoped that economic impact and similar studies will fill this factual gap in case preparation. Mr. Watson Bowes, MAI, state

[3] The references cited by the italic numbers in parentheses are listed beginning on page 33.

this proposition in the following way: "Economic studies are not only advantageous in appraising highway right-of-ways but they are absolutely necessary. Every highway department appraiser employs such studies to some extent. Some appraisers relate economic studies to the subject properties by making only mental notes as they are developing their estimates of fair compensation. Such mental notes are difficult to transmit to juries and do not show on any appraisal report so they can be used as a negotiating tool by the negotiator" (*9*).

An operating official in the highway field of the State of Washington believes these studies serve: ". . . to provide data to staff and fee appraisers to assist them in more accurately measuring the just compensation in a partial taking problem. I believe that in all States the right-of-way divisions are finding that the constant improvement in appraisal techniques is resulting in more and more accuracy in the appraisal of a total taking, or in the before value of a property involving a partial taking" (*10*).

Right-of-way specialists generally are in favor of using such land value studies in appraisals as well as in court proceedings. Frank C. Balfour, Executive Vice Chairman of the American Right of Way Association; Rudolph Hess, Chief Right of Way Agent, California Division of Highways; Leonard I. Lindas, Chief Counsel, Oregon State Highway Department; Victor Eichhorn, Director of Right of Way Division, Michigan State Highway Department; and many State highway officials; and the Bureau of Public Roads have indicated their support of such research (*11*).

Land value studies of impact in an area that abuts a highway, compared with one for an area that is not near the highway improvement, would be particularly pertinent in court cases where land value trends are necessary factual background for the jury. It then would be possible for the court and jury to evaluate the work product of the expert appraiser against these data. The California Law Revision Commission (*12*) has seen fit to recommend the use of appraisal theory in condemnation cases in order to systematize the concepts of valuation and the background of testifiers. It usually is within the court's power to determine the rules of compensation, as a means of implementing a legislative decision, for the taking of a specific property subject to a public need.

In the main, empirical evidence at the trial consists of the appraiser's opinion of market value, as described in the following statement: "An appraiser is supposed to reject elements that are remote, fanciful, speculative, and uncertain. In judging the situation the appraiser must determine whether the facts establish a diminution in value with reasonable certainty, as distinguished from merely hypothetical or fanciful assertions having no effect upon value (*13*).

Yet, in a world where statistical data are used to ascertain and provide decision-making tools to management and government, it would appear most proper to prove the appraiser's opinion of value by empirical or statistical evidence. With the time-honored means of introducing appraisal opinion as expert testimony, the highway lawyers have avoided the use of much significant information, such as the various land value studies. The implications of these studies to evidentiary presentation will be discussed in a later part of this article that deals with the admissibility of research evidence in highway right-of-way litigation.

Interchange Impact

Right-of-way personnel have been especially interested in the subject of the impact of interchanges on land values: experience thus far has shown these interchanges to be the hubs of economic activity. Such impact studies at interchange points tend to provide data that can be utilized to indicate objectively the after value of property at such points. In the State of Washington, a number of case studies at interchange points have been prepared that may be used for such a purpose. In addition to this type of case study, general land value trends at interchange points are being established in the economic impact studies at the University of Washington, Texas A & M, and in the States of Michigan, Minnesota, Ohio, Oklahoma, Pennsylvania, and Washington, and many others where interchange impact on an area (general influence on land values) and on specific parcels can be delineated.

Severance Damage Studies

A major means of establishing property value is by use of the comparable parcel as the standard of value, so that the court and the jury may have the opportunity to approximate true value. To aid the court in this factfinding function, right-of-way personnel in State highway departments and private appraisers, through such professional organizations as the American Right of Way Association and various appraisal societies, have interested themselves in establishing various research programs in land values. By such land value studies an attempt is being made to supply the appraiser with knowledge of the economy in which he operates. The findings in such studies provide him, and other persons involved in right-of-way or eminent domain proceedings, with the reaction of land values in situations similar to the one involving the parcel in question. The important contribution made by this research is the aid that it may provide for valuing partial takes. Where whole parcels are acquired by condemnation, the establishment of market value is not as difficult as it is where only part of a parcel is acquired and the damages to the remainder must be evaluated.

Nature of severance damage studies

Severance damage studies are intended to facilitate the objective determination of the effect that the partial taking of a property has upon the value of the remainder parcel. Such information is invaluable if each affected property owner is to be reimbursed not only for property taken but also for damages to the remainder.

To measure the effects of a partial taking of property, most severance damage studies rely on a before-and-after approach—the value of the property before the highway taking compared with the total amount that the owner received from the property; for example, for property taken, for damages to the remainder, and for the sale of the remainder. Ideally, the adjustment that should be made with a property owner is the difference in the fair market value of the entire tract before the taking, and the fair market value of the remaining real property after the taking. While the appraisal of these before-and-after values is made at the same time, the appraiser must attempt to determine the value as of two different times—one in the past before the highway, and the other in the future after the highway has been constructed and its influence felt. In those situations where the remainder is sold so that a reliable indication of the value of the remainder is provided, the elements for a meaningful comparison are available—the original value (determined by recognized appraising techniques) versus the value realized by the owner (total payments for property taken, for damages, and for remainder parcels). If there is wide discrepancy between these two amounts, either too much or too little is being paid for right-of-way property or damages; the legal limits to these rules are described later. Appendix V is a list of severance damage studies, completed or in progress.

Similarities Between Severance Damage and Economic Impact Studies

Severance damage or partial taking studies and economic impact studies have several similarities, and either type of study may sometimes be referred to generally as a land economic study. In fact, severance damage studies may be considered a particular type of economic impact study. For example, in a land value study now underway in Colorado, special emphasis is being given to the analysis of severance damages related to controlled-access highways. In general, severance damage studies and economic impact studies are alike in that the identification and measurement of effects that can be traced to highway improvements are sought. The careful attention given in both types of studies to measuring the impact of highways that have been built in the past results from a common objective—the development of a factual basis for predicting highway effects.

Some Contrasts Between Severance Damage and Economic Impact Studies

The differences between severance damage and economic impact studies result primarily from the identification sought for the different types of benefits. Both types of studies ordinarily consist of a comparison of the situation before and after the highway, to

determine the effect of the highway. Economic impact studies ordinarily are concerned with identification of benefits or disadvantages that accrue to an entire community or some portion of a community. General effects can be defined as injuries or benefits that the owner sustains or receives in common with the community generally and that are not peculiar to him (*14*). For example, the increment in land values that a community may experience from a bypass route would be termed a general benefit. The legal basis for these distinctions are described later in this article.

In severance damage studies, the concern is with highway effects on particular land parcels taken in part for highway property. If the total amount received by a property owner for right-of-way, for damage to the remainder, and from sale of the remainder exceeds the value of the property prior to the highway, a benefit has accrued to the owner. Often of great concern in severance damage studies are the special benefits—the highway effects that accrue to a particular land parcel taken in part for highway right-of-way—that are peculiar to a specific property and that are not shared by other properties in the community. Although the distinction between special and general benefits often becomes blurred, it is more common to look to special benefits than to general benefits in offsetting the damage suffered by remaining land parcels or in paying for property acquired in part.

Different approaches

Whether the focus of a study is on general or special benefits obviously affects the method used to identify these benefits. The case study approach commonly has been employed for severance damage studies. The experience of a highway-affected community with respect to land value trends, business volumes, or employment has been of more concern in the economic impact studies. The different types of control areas used in severance damage studies and in economic impact studies also reflect the different emphasis of these studies.

A fundamental type of analysis in economic impact studies, in which the detection and measurement of general benefits are sought, involves comparison of an area influenced by a highway and an area removed from highway influence. The ideal control area is one that is like the study area except that it has a complete absence of highway effect. The types of controls that have been used in severance damage studies show some variation; this, no doubt, has been caused primarily by differences in State laws as to which benefits can be considered in the establishment of compensation. In States where both general and special benefits can be applied against the cost of acquiring right-of-way property, a control area removed from the highway influence is desirable. However, in more than half the States where only special benefits are to be considered in the determination of adjustments to be made with affected property owners, control areas are needed in the immediate neighborhood of the study parcel. A representation of how measurement of general and special benefits is sought by severance damage studies is included in appendix IV.

Shortage of Factual Information

The lack of systematically organized information poses a major problem in the evaluation of partial takings. The severance damage program of the Bureau of Public Roads and the State highway departments, for which an important goal is obtaining a basis for use in countering extravagant property damage claims with objective analysis, is expected to provide such information.

The difficulty of gaging highway effect without careful reference to the experience in comparable situations often has been illustrated. A severance damage study in Michigan, for example, revealed that a highway-influenced parcel of land, which was expected to have a value of only 5 percent of what it was worth prior to the highway, in fact, had an after value of 115 percent of its before value (*15*). In Ohio, researchers have noted that there has been no instance of an owner having had to sell a remainder parcel for as little as 10 percent of its former value, although estimates of 90-percent damages for landlocked property are reported to have become fairly common (*16*).

The dearth of factual information about what happens to remainder properties is widely recognized. One account of a typical appraisal states that it contains ". . . solid proof on the before value of the lands and improvements with full documentation . . ." and then the frustrating words, ". . . in my opinion the remainder is damaged 50 percent by reason of proximity" (*9*). Many appraisers are keenly aware of the fact that large scale right-of-way appraisal for highway improvements is relatively new, that it poses new problems, and that it intensifies the need for factual information as to how the market reacts to remaining portions, in order to make the after estimates something more than guesses from a crystal ball (*17*).

Accomplishments from Severance Damage Studies

The goal for severance damage studies—to make it possible to predict at the time of right-of-way taking what effect the highway will have on the remaining parcel—appears to be almost insurmountable. Any two or more properties obviously vary to some degree so that predictions of what may happen to one piece of property, on the basis of the experience with other properties, can be only approximate and must be made with considerable caution. The difficult nature of this problem is emphasized by the wide variation in the experience with remainder parcels; the unit value of remainder parcels in one study ranged from one-half to nearly 15 times the former value (*18*).

While exact evaluation of benefits and damages associated with right-of-way taking is desirable, real benefit can be derived from severance damage studies that are short of such precision. This is evidenced by the usefulness of current studies for such purposes as right-of-way acquisition, public relations, highway planning, and administration. In instances where benefits associated with a highway right-of-way taking exceed the value of property taken, it may be sufficient merely to determine that benefits exceed or equal the value of the property taken; benefits ordinarily cannot be offset unless they are matched by the value of right-of-way acquired or by damages, as described later in the legal analyses. Determination of the exact amount would appear to be necessary only where the benefit is insufficient to offset costs—in situations where the amount of the award due an owner exceeds any benefits that may be allowed and, therefore, where the amount of the award payable depends upon the establishment of amounts for benefits or for any damages. Consideration of benefits associated with partial takings, without the assignment of exact benefit amounts, apparently can be of some usefulness in those situations where no attention whatsoever is now being paid to this important element of the post-highway situation. In such cases, recognition of any highway benefits should be helpful in establishing more reasonable costs for right-of-way acquisition.

Market approach

Partial taking studies in which benefits merely are recognized or estimated in a general way—the benefits approach—have considerable usefulness. However, in a number of studies the scope has been increased and a market approach applied—a measure of effect determined by the market place. These studies have provided information for comparisons of estimated and actual highway effects on remainder parcels, and some of these estimates have been shown to be fairly accurate. In several instances, estimated and actual damages, generally established by actual sales prices, have been found to be within a few percentage points of one another; for example, estimated damages of 13 percent and actual damages of 7 percent; estimated damages of 37 percent and actual damages of 2 percent; and estimated damages of 80 percent and actual damages of 70 percent (*19*).

The progress made in these studies in evaluating the effect of certain factors, such as having more than one potential purchaser for certain types of remainder parcels, should be of general usefulness in determining what may happen to remainder parcels. As might be expected, the selling price of landlocked parcels with two or more potential buyers has been found to be significantly higher than that for remainder properties with only one potential buyer; that is, one abutting owner. Certain parcels in Ohio with one abutting owner were found to have a percentage of recovery of 20 percent (that is, the selling price in the after period was 20 percent of the appraised value before the highway), while parcels with two or more abutting owners had a recovery rate of about 80 percent (*20*). In Michiga

recovery rates for parcels with one potential buyer were found to range from 32 to 57 percent of the former value and from 90 to 113 percent for remainder parcels with two or more possible purchasers (*15*).

Research in partial taking studies also has provided assistance in forecasting highway effects by relating experience with remainder parcels to such factors as size of remaining parcel, size of farm unit before the severance, and type of potential purchase. For example, the degree of damage sustained by a 160-acre farm from the loss of 5 to 10 acres is less than that experienced by an 80 acre farm from a similar loss. In at least one investigation, small remainder parcels were shown to have a lower rate of recovery than larger parcels, although preliminary findings from another investigation showed no apparent correlation (*19*).

The preceding discussion indicates the help that these severance damage studies can afford by providing authoritative proof of just what can be expected to happen to the value of the remainder parcel of land. Another accomplishment obtained from these studies is a sign that progress is being made in the objective determination of highway effects. This is indicated by the apparent increase in the expectation that appraisals of after values are to be supported by specific written justification, and the need for adequately supported appraisals.

Collection of Data on Severed Parcels

To develop a file or bank of cases from which information on comparable sales experience for severed parcels may be obtained, the Bureau of Public Roads and State highway departments are sponsoring the use of a standardized study procedure, which encompasses a manual of procedures and suggested techniques. Use of this instructional material will make published results and methods available to highway departments; this information can be helpful in solving controversies on valuation. The necessity for such a bank of information exists because experts, appraisers, and others valuing property generally do so in terms of whole parcels; real estate valuation experience is most prevalent in the area of whole parcels. Little analyzed data are available for use in determining the experience with severed parcels or partial takes. This bank will furnish the means for developing comparable sales information for severed parcels, according to the procedures designated by the manual for obtaining this factual information. The manual also provides suggestions for obtaining maximum usefulness from a uniform severance damage form, which has been developed by the cooperative efforts of interested persons in the various States, the American Right of Way Association, and the Bureau of Public Roads.

The recommended procedures for using the severance damage data include a description of the way in which severance damage data are to be collected and processed, and a brief description of the types of analyses that will be feasible for mechanizing the recording and sorting of these data. It is hoped that a central bank of information regarding similar severed parcels will be available at the Bureau of Public Roads for the use of State officials. With the mechanical sorting devices to be used, it will become possible to make comparable sales information available to researchers and appraisers in the field.

The uniform schedule form used (PR 1030) requests the following data: General information on parcel location, type of highway, and type of access; description of tract, parcel taken, and remainder tract; relevant data on taking, size and use of parcel, zoning, visibility, elevation, appraisal value, compensation, court awards, and subsequent sales. All of the elements of information requested on this form are relevant to the establishment of parcel comparability and market value. The details requested on these forms should make it possible to narrow down the comparabilities so that the evaluation of the parcels may be comprehensible to factfinding bodies.

Because of the systematic nature of the collection of severance damage cases, it is expected that comparable sales information will be made available readily and that courtroom presentation will be facilitated. The comparability still will need to be proved and still will be subject to dispute, but standardization of procedure may eventually determine the use of these bank cases in all States.

Evidence for Offsets

These severance damage studies and case histories of individual severed parcels also will be used to indicate the amount of benefits as offsets to damages and the value of property taken in cases where such offsets are permitted. Where benefits may be offset, difficulty usually is encountered in establishing the amount of general or special benefits. Use of these bank cases also is expected to make it possible for appraisers to estimate the amount of benefits; estimates based on elements that historically have been associated with similar cases.

Despite the case histories that will be made available from these studies, the usual objection is expected to be that the amount of offset to damages, if any, found in such studies does not apply to the parcel under consideration because the studies represent property that is different in nature, type, location, etc., from the property under litigation. In the case of the severed parcels, it is hoped that the histories of land parcels of similar types, as valued through market sales, will be indicative of the value of the parcel in question. The utilization of land value studies in court work generally will be objected to on the grounds that, as averages, the studies represent a dispersion of properties, that they deal with hearsay, and that they do not represent the property involved. In the next two sections of this article, the current status of the law of proof, and the possibilities of meeting the hearsay and other objections are discussed. These discussions point the way toward more effective utilization of such economic studies in the courts, and they also point out the limitations to the use of such studies.

PERTINENT LAWS OF EMINENT DOMAIN AND EVIDENCE

Introduction

The possible uses of economic evidence in highway condemnation litigation must be considered in light of the pertinent law of eminent domain and evidence, and in light of the problems involved in applying this law. With this objective, the following points are examined: (1) The nature of the condemning authority's duty to compensate those whose property it takes. (2) The rules and criteria of value whereby this compensation is measured, particularly where only part of a tract of real estate is taken for highway purposes. (3) The types of permissible evidence currently used to prove the property value, damages, and benefits designated by these rules and criteria of value as components of the property owner's just compensation.

In this examination of the current law, how the types of economic evidence heretofore discussed would meet existing inadequacies and thereby facilitate a more accurate measurement of the property owner's just compensation are considered, as well as how these types of economic evidence would comply with the rules of evidence. The potential challenges raised by the rules of evidence to the admissibility of this economic evidence are discussed in another section of this article.

Constitutional Provisions for Just Compensation

Fundamental to the law of eminent domain in the United States and in the several States are the Federal constitutional provisions (*21*) that require just compensation to be paid to owners of private property taken pursuant to the power of eminent domain. These Federal provisions and a majority of the State constitutions (appendix I, part A) require only that such compensation be made to the owner of property taken by eminent domain, but some State constitutions (appendix I, part B) extend the right of just compensation to owners of property damaged by the exercise of eminent domain. Taking, in the constitutional sense, entails either actually entering on the land or depriving the owner of substantially all beneficial use of the property (*22*). Damaging, in the constitutional sense, occurs when noncondemned land either has sustained an injury actionable at common law or has been injured more than the general public by the physical disturbance of one of the owner's rights therein (*23*). Compensation for taking is measured by the property's value at the time of taking, as defined (*24*); compensation for damaging is measured by the value depreciation of the damaged property (*25*). Thus, the exercise of the eminent domain power requires ascertaining in every instance the value of the property taken and, in some instances, both the value of the property taken and the value depreciation of property not taken.

Value Criteria for Measuring Just Compensation

The judiciary, whose exclusive function it is to determine the exact amount of compensation in each case, has largely formulated the rules and criteria of evaluation in eminent domain cases. Courts generally have held the just compensation constitutionally required for the taking of property to be the property's value at the time of the taking (*26*). This value has been further defined, in most cases, as the property's market value (*27*); but where market value has not been reasonably ascertainable, courts have had recourse to what they term the actual or intrinsic value of the property (*28*).

The concept of market value (*29*), especially as applied to real estate, does not readily admit of concise and practical definition. The practical application of this concept in condemnation proceedings has led to the generally accepted definition of market value as: The amount of money that a purchaser willing but not obliged to buy the property would pay to an owner willing but not obliged to sell it, taking into consideration all uses for which the land was adapted and that might in reason be applied (*30*).

The constitutional requirement of just compensation for taking, from which stems the market value criterion, implies full indemnity to the owner (*31*). However, this indemnification extends only to the value of the property taken and does not guarantee that the owner will receive a return for his investment in the land (*32*). For purposes of establishing market value, the land is looked upon merely as so much land and apart from its sentimental value to the owner or his willingness or unwillingness to sell it (*33*).

Market value criterion

Under the market value criterion for establishing just compensation, all the elements of value that contribute to the saleable character of the land are relevant; that is, all the facts that an owner would naturally and properly press upon a prospective buyer's attention, and that naturally would influence an ordinarily prudent person desiring to purchase (*24*). Thus, the owner of condemned land is entitled to have it evaluated in light of the highest and best use to which the land can reasonably be adapted, irrespective of its current use or the owner's immediate plans for its use (*35*). However, only the highest and best uses legally permissible are those that are not remote or speculative and that would affect the present market value of the land (*36*).

The market value criterion has been bypassed in certain types of cases. The condemned land may have been improved and adapted for such a special usage as not to be readily saleable at anything near its real value (*37*) or other circumstances may preclude the ascertainment of market value (*38*). The landowner's constitutional right to full indemnity for the loss has led courts, under these circumstances, to adopt the intrinsic value or value-to-the-owner criterion (*39*). Under this criterion, the objective value of the property, to the owner or to anyone else for any special use to which it has been adapted, is considered (*40*). It is frequently arrived at by calculating the replacement cost of the improvements on the land, less depreciation, plus the value of the land (*41*).

Only the value of the property taken is to be indemnified under the just compensation for taking provisions of the Federal and State constitutions. Thus, the value of the buildings and fixtures thereon are properly compensable (*42*). However, any business operated on such property, including any goodwill appurtenant thereto and any anticipated profits therefrom, is not considered a property right within the meaning of these constitutional provisions (*43*). The courts have reasoned that the business is severable and distinct from the land; only where the business is taken over by the condemning authority will the owner be compensated for the value of the business (*44*). The impact of this rule has been alleviated in many States. In some, special legislation authorizes compensation for the taking or damaging of a business by eminent domain (*45*). In others, the courts have admitted the profits of a going business concern on condemned property as evidence of its value for its highest and best use (*46*). The restricted scope of this indemnification also precludes compensation for the frustration of contracts related to the condemned property (*47*) and for the inconvenience and expense incident to being dispossessed of the property (*48*).

Modifications in Partial Taking Cases

Computing just compensation for the condemnation of only a part of a tract of land raises a new series of valuation problems. Even under a mere taking provision, the owner of a tract of land is not confined to recovery of the value of the land taken. Besides any increased value that may inhere in the land taken because it is part of a larger tract (*49*), the owner of such a partially condemned tract further is entitled to recover any severance damage to the remainder (*50*). These severance damages include the correlative loss of any value that may have inhered in the remainder as part of the larger tract (*51*). They further include any present or prospective depreciation in the remainder's market value that naturally and proximately results from the proposed use of the condemned part. Any aspect of the proposed use that may detrimentally influence a prospective purchaser of the remainder property is considered in ascertaining these damages (*52*). Severance damage studies, insofar as they trace the subsequent market-value history of land severed for highway construction, are especially designed to be of assistance in the computation of severance damages.

Setoff of benefits

Integrated with the assessment of severance damages are the various rules governing the setoff of benefits accruing to the remainder parcel from the prospective use for which the land has been condemned. The benefits that frequently accrue from a public improvement to neighboring lands are often the subject of a special assessment on the neighboring land thereby benefited (*53*). Such assessments, which serve to defray or cover the cost of the public improvement, are a proper exercise of the power of taxation (*54*). In most jurisdictions, with various limitations hereinafter discussed, the same type of benefits are considered in computing the compensation due a landowner for land partially taken by eminent domain. Accordingly, prospective benefits, which will enhance the market value of land from which condemned land has been severed and that are attributable to the particular public improvement for which the condemnation has been made (*55*), have been setoff against the compensation to which the landowner would otherwise be entitled.

Benefits have been classified as either special or general. Special benefits accrue in a peculiar way to a particular tract because of its direct relation to the public improvement. Conversely, general benefits accrue to the general public of the community as well as to directly related lands. In highway condemnation cases, courts usually have distinguished between these two types of benefits on the basis of whether they accrue only to lands abutting the highway or to nonabutting lands as well (*56*). Thus benefits that accrue to nearby lands, which do not abut the highway, are regarded as general benefits (*57*). However, benefits accruing to abutting lands, including land of which no part has been taken and land that has been partially condemned, are regarded as special benefits (*58*). Severance damage studies, which analyze the market value development of highway severed remainder parcels, have been designed to assist in the determination and measurement of special and general benefits. With respect to general benefits, however, a similar service may be provided by other economic impact studies, which analyze the market value development of communities affected by highway development.

Both general and special benefits may be setoff in some States, but only special benefits are deductible in a majority of States, as shown in Parts A and C of appendix II. Setoff of both types has been held perfectly consistent with the property owner's right of full indemnification and, accordingly, has been constitutionally sanctioned (*59*). Historically, setoff has been justified as an exercise of the power of taxation (*60*). Setoff of general benefits has been disallowed primarily for two reasons. Such setoff has been regarded as an unjustly exacted payment from the owner of partially condemned property for benefits equally enjoyed by his neighbor without charge (*61*), and general benefits also have been regarded as too speculative to be assessable as compensation (*62*).

Rules of setoff

The rules of setoff further differ as to the elements of compensation from which benefits may be deducted. Except in two States, benefits are setoff against severance damage

to the remainder, as shown in appendix II. Under this rule, benefits are regarded as one of the elements enhancing the property's market value, only the depreciation of market value is compensable (*63*). On the other hand, several States prohibit the setoff of benefits against the value of the property taken; these States are identified in appendix II. A primary requirement of many of these State constitutions is that just compensation be made in money, which precludes setoff against the value of the land taken (*64*). Conversely, where setoff against the full compensation is allowed, it is regarded as the only just allocation of cost between the public treasury and the private property owner (*65*).

Thus, in cases where part of a tract of land is taken by eminent domain, determination of the landowner's just compensation will be affected by the rules of setoff in any one of five different ways, depending on the local law: (1) General and special benefits may be setoff against both the value of the land taken and the severance damages to the remainder; (2) general and special benefits may be setoff only against the severance damages; (3) only special benefits may be setoff against the value of the land taken and the severance damages; (4) only special benefits may be setoff against the severance damages; or (5) no benefits of any kind may be setoff. Refer to appendix II for information on setoff rules. The cost of highway right-of-way acquisition, which necessarily involves much partial taking, is substantially affected by whichever setoff rule applies. This is illustrated by the varying amounts payable in a hypothetical situation shown in table 1, in which the original value of the property was $200,000.

Formulas for Computing Just Compensation

The several considerations incident to ascertaining the condemnee's just compensation in partial taking cases have resulted in two judicially created rules. Under the before-and-after method, the condemnation tribunal always appraises, according to the same principles heretofore discussed, the value of the entire tract of land before the partial taking (*66*). Then, if no benefits are to be setoff, it appraises the remainder without regard to any expected benefits (*67*). However, if any benefits can be considered, the remainder is appraised in light of those anticipated benefits that properly can be setoff (*68*). The inherent shortcoming in use of this formula lies in the inability to segregate the value of the part taken from the value of severance damages to the remainder. For this reason, it would seem inadequate where benefits are to be setoff only against severance damages. Although the before value may be shown by comparable sales of similar property, the very nature of the severed remainder, which abuts the highway after the taking, may severely limit the availability of comparable sales evidence of the after value. However, severance damage studies, through the classification and compilation of the sales of similarly severed parcels, would provide such needed comparable sales evidence.

Value-plus-damages formula

As an alternative, the value-plus-damages formula provides a much more complex but theoretically precise method of computing the condemnee's award in partial taking cases. Under this formula, the value of the part taken is appraised separately (*69*); then the severance damages to the remainder are determined either as a separate sum (*70*) or in light of the benefits properly setoff (*71*). Where the damages have been separately computed, any permissible setoff benefits are assessed (*72*). Then, the final award is computed by subtracting all properly setoff and properly assessed benefits (*73*) from the sum of the value of the land taken and the severance damages or, as determined by local law, only from the severance damages. Severance damage studies, by focusing on the subsequent history of severed parcels, are especially geared to provide reliable indices on severed remainders for both the damages and benefits resulting from highway takings.

In a comparative appraisal, each of these formulas appears to have its own distinctive merits. Only the value-plus-damages rule recognizes and theoretically complies with the condemnee's constitutional and statutory rights to be compensated in money for land taken. However, the artificial and complex division of this formula make it inherently difficult to apply. Under it, the same element of damage may be assessed in duplicate under different theoretical guises (*74*). On the other hand, the before-and-after rule stands out for its simplicity of application and its inherent capacity to reflect the value of the land taken and the severance damages. As noted, the main drawback to use of this rule is the inability to segregate the value of land taken from severance damages, in order to assure compensation in money for the former.

Table 1.—Example of variation of compensation for property according to provisions for offsetting benefits against damages

L=$80,000=value of land taken.	Bs=$40,000=special benefit.
S=$20,000=severance damage to the remainder.	Bg=$50,000=general benefit.

Prevailing rule	Calculation	Compensation due owner
1. In some States, both special and general benefits can be used to offset both land cost and severance damage.	(L+S)−(Bs+Bg)	$10,000
2. In some States, both special and general benefits can be used to offset severance damage only.	L+[S−(Bs+Bg)]	80,000
3. In some States, special benefits only can be used to offset both land cost and severance damage.	(L+S)−Bs	60,000
4. In a number of States, special benefits only can be used to offset severance damage only.	L+(S−Bs)	80,000
5. In two States, no offset of benefits is permitted	L+S	100,000

PROOF OF VALUE

Applicability of the Rules of Evidence

The concepts of value relating to just compensation and the formulas integrating them can be effectuated only insofar as the rules of evidence permit. Each time the power of eminent domain is exercised to take private property, the *quantum* of the owner's just compensation must be determined by an arbiter of the facts. State constitutions and statutes variously provide for this function to be performed by either a board of commissioners (*75*) or a common law jury (*76*). The board of commissioners, chosen for its peculiar skill and knowledge in property valuation, generally is not bound by the rules of evidence (*77*). Rather, its members are simply charged to appraise, impartially and to the best of their skill and knowledge, according to the substantive rules of valuation (*78*). In contrast, the common law jury, not chosen for any special knowledge or skill, is for the most part bound by rules of evidence (*79*). Hence, in proof of the condemned property's value, the rules of evidence exclude from the jury's consideration any evidence that is not both competent in itself and material and relevant to this issue (*80*). Severance damage studies, by the very nature of their subject matter, ought to be found both material and relevant to the issue of market value in partial taking cases; the evidentiary status of these studies might be challenged on the basis of competency. However, as hereinafter discussed, properly conducted severance damage studies ought to be found sufficiently competent to be admitted as an exception to the hearsay rule, which is discussed in connection with the admissibility and use of research evidence.

The Objective of Market Value Evidence

Within the scope of these rules, certain types of evidence are commonly and strategically used in proving the market value of condemned property. Market value is not simply an inherent quality of the property; it is largely a reflection of the state of mind of the public with respect to that property (*81*). This state of mind commonly is proved by the opinions of qualified witnesses, who testify as to what value they estimate the public would attach to the particular property taken or damaged by eminent domain. This state of mind also is frequently proved by deduction from the prices paid in recent sales for the same or similar property, which are admitted

as evidence of the market value. By virtue of the index of this state of the public mind, which the severance damage studies have been designed to provide, these studies ought to implement opinion testimony on the market value issue and provide a broader scope of sales evidence.

Evidence of Other Sales

Actual sales of the condemned property not too remote from the valuation date, when the property has been voluntarily bargained for in good faith, are admissible evidence of the property's market value (*82*). Presentation of this type of evidence is most strategic. Any prospective purchaser of land is bound to be influenced by the price recently paid for it in a voluntary and bona fide sale. The jury seeking to indemnify a property owner for his loss, naturally, is influenced by any price recently paid by him for the property. The mere fact that it is the same property precludes many of the distracting collateral issues that otherwise would arise (*83*). Nevertheless, such evidence is not conclusive of the property's value at the time of taking (*84*).

In contrast to evidence of recent sales of the condemned property, evidence of recent sales of property similar to the condemned land usually is more readily available but less readily admissible. Although evidence of such sales is universally admissible to cross-examine opinion testimony (*85*), only a minority of four States now prohibits it use as direct evidence of market value. Refer to appendix III for more information. However, the law of the majority of States favors the soundness of admitting such sales as direct evidence of market value. Thirty States expressly allow such sales as independent evidence of market value; those of the remaining States, whose courts have considered this kind of evidence, all tend to give it some affirmative probative value, and recently, in a noticeable changeover, a number of former minority States have adopted the majority rule (*86*).

Very cogent reasons support the admissibility of such sales, either as independent evidence of market value or in support of opinion testimony. Market value, the criterion of just compensation, is the price at which property sells in the open market. When made under normal and fair conditions, such sales are by their very nature a more valid indication of market value than the speculative opinions of witnesses (*87*). Thus, when offered in support of such testimony, sales evidence enhances the testimony and, when offered as independent evidence, provides a firm basis for any condemnation award for which other kinds of evidence may be ignored. Severance damage studies, insofar as they are based on comparable sales of severed parcels, ought to be accorded equally strong probative value.

Inherent drawback

The inherent drawback to use of recent sales of similar property as evidence is the multitude of collateral issues that each such sale raises. For each such sale proffered in evidence, the court often decides as preliminary questions of fact the numerous issues of comparability, proximity, and voluntariness, hereinafter discussed. Furthermore, for each such sale admitted in evidence, the jury must decide wherein and to what extent the recently sold parcel differs from the condemned parcel and must make allowance for such differences in arriving at the latter's value (*88*). The multitude of these collateral issues by their digressive effect, especially when multiplied by the number of comparable sales introduced, may substantially impede the valuation procedure. For this reason, the number of comparable sales admissible in any one case may be regulated by the court (*89*). As heretofore discussed, the admission of statistical surveys would require the court to determine, as a preliminary question of fact, whether the proper methodology had been followed in conducting the survey to establish its reliability as evidence. However, once this collateral issue had been resolved, a much broader scope of comparable sales evidence would be available to the jury.

Restrictions

Certain requirements of similarity and proximity restrict the admission of all sales of similar property. The property sold must be sufficiently similar in character and geographically proximate to the condemned property to be useful in reflecting the latter's market value (*90*). The exact degree of each qualification required in each case is determined largely by the trial court within its discretionary power (*90*). However, certain elements of similarity are almost universally demanded by the courts. Where nearness to schools, churches, transportation, and shopping centers substantially influences the value of property, only sales of property located a similar distance from these public facilities may be admissible as comparable (*91*). Where the highest and best use of a tract of land is for agricultural purposes, sales of more distant property with soil of a similar character may be deemed sufficiently similar to be admissible (*92*).

Where the condemned property has been adaptable for such a special highest and best use that sales of similarly adaptable property in the same community were not available, the requirement of geographical proximity has been largely abrogated (*93*). For the same reason, the market value of severed lands with a special highest and best use because of their adjacency and access to a major highway ought to be provable by the sales price of a comparable remainder in another community. Similarity in the topographical features, size, and shape of the two parcels also ought to be considered (*94*). If the individual sales compiled in severance damage studies were to be introduced in evidence, each such sale would be subjected to these same tests of comparability. However, if a survey of such sales made in a severance damage study were to be admitted in aggregate form, the comparability of the sales included would be shown by an examination of the methodology and criteria used by those who conducted the survey.

Timeliness

Furthermore, to be admissible as evidence, sales of similar land must be so proximate in time to the date when the condemned property was taken as to furnish an indication of value at the latter date (*95*). The permissible interval depends partly on the stability of market conditions and the availability of more recent sales but ultimately, in each case, it is determinable by the court within its broad discretionary power (*95*). The timeliness of sales included in severance damage study surveys easily could be shown by the survey director's testimony.

Both recent sales of the condemned property and recent sales of similar property, in order to be admissible, must have been voluntary and the property bargained for in good faith (*96*). The requirement of voluntariness precludes admission of evidence of sales wherein either party acted under any coercion. Thus, where the threat of condemnation or the need to sell out or purchase with undue haste has induced either party to consummate a sale, such a sale is not admissible evidence (*97*). On this basis, a majority of the States exclude all sales to a condemnor or purchaser with the power of eminent domain (*96*). To assure that such sales evidence reflects market value, only such sales as were bargained in good faith by both parties are admissible (*76*). Thus, only sales made by parties capable and desirous of protecting their own interests are admissible. The voluntary and good faith nature of sales included in severance damage studies is one of the facts ascertained by those who conduct such studies. Thus, the exact degree of voluntariness and good faith common to all such sales included in any survey introduced in evidence could be ascertained by examining those who conducted the survey.

Evaluating sales evidence

When a sale of similar property has been ruled admissible, it is merely deemed sufficiently similar to be helpful in evaluating the condemned property. Both parties are then entitled to introduce evidence of the differences between the two properties to show wherein and to what extent the condemned property's value is greater or lesser (*99*). A severance damage study survey, when ruled admissible, also might be shown on examination of the study director to have been based partially on dissimilar sales. Thus, the admission in evidence of such a survey would not preclude either party from showing wherein the condemned property's value should not be governed by the survey. However, severance damage studies by the breadth of their scope would weigh heavily against any speculative valuation of the condemned property.

When such sales are admitted as independent evidence of value, the sales price must be proven with as much formality as other material facts. Thus, those considered competent

to testify to the prices paid in such sales are persons who were parties or brokers to such sales or who in some other manner knew of the price paid of their own knowledge (*100*). Accordingly, the mere recital of consideration in a deed and other hearsay sources of price information are not admissible (*101*). However, the Federal revenue stamps affixed to real estate deeds have been admitted as evidence of the amount of consideration (*102*). Thus, the sales price information collected in severance damage studies would be reliable and, therefore, competent evidence so long as, in the conduct of the study, this information was taken either from interviews with parties to the transactions or, in some States, from the Federal revenue stamps on the deeds.

Opinion Evidence

Historically, market value has been regarded by the courts as merely a matter of opinion (*103*). To assist the condemnation jury in forming its opinion of the market value of property taken or damaged by the exercise of eminent domain, the opinion testimony of those with special knowledge relating to the property's value is admissible evidence (*104*). Such opinion evidence, however, is merely advisory and, accordingly, not binding on the jury (*105*).

Consistent with the rationale for the admissibility of all opinion testimony, such opinions may be given only by those possessed of some special knowledge or skill deemed valuable to the jury in forming its conclusion (*106*). In condemnation proceedings, real estate experts are considered competent everywhere to give opinion testimony on the property's market value (*107*), and in some States neighboring residents and businessmen also are considered competent to so testify (*108*). Moreover, in addition to their respective special knowledge or skills, all condemnation value opinion witnesses must possess certain factual knowledge. All must be both personally acquainted with the condemned property and personally familiar with the state of the market in that area (*109*).

Those who have bought and sold, valued, or managed real estate in the community are deemed to have acquired therefrom such skill in appraisal and such knowledge of property values as to be real estate experts competent to give opinion testimony (*107*). Such experts also must have a personal knowledge of the condemned property and market conditions in the area. Accordingly, they must base their testimony on characteristics and conditions that they actually have observed rather than on hypothetical conditions (*110*). Only in the absence of a market value are specialized experts competent to give opinion testimony regarding the property's intrinsic value (*111*). Real estate expert testimony has been regarded as the most practical medium of presenting to the jury the appraisal hypotheses on which either party seeks to have the condemnation award based (*112*). Both severance damage studies and other economic impact studies would provide a means of testing such a witness's expertise in appraising property affected by a highway.

Lay witnesses

In a majority of jurisdictions, neighboring residents and businessmen are deemed competent to give opinion testimony (*113*). This competence is premised on the special familiarity with local real estate values that they are presumed to have acquired by their long-standing activity and interest in the area (*114*). Such lay witnesses are not deemed to possess any special appraisal skill; rather, it is their special familiarity with local values that qualifies them to give value opinion testimony (*115*).

The speculative nature of such testimony perhaps is best illustrated by a recent Missouri highway condemnation case (*116*) in which the only opinion witnesses on value were two neighboring farmers. An award of $400 was determined by commissioners. Both parties appealed to the circuit court for a jury trial. On the before-and-after basis, one farmer's testimony would have warranted an award of $4,725, and the other's testimony, an award of $2,500. Apparently influenced by these lay witnesses, the jury awarded $2,000. Where no real estate experts who are familiar with the condemned property and values in its surrounding area are available, use of such lay witnesses may be the only available means of proving value. In such a situation, severance damage studies ought to provide both a ready selection of sales of comparably severed parcels and a more reliable index of the remainder's value through survey evidence. Furthermore, economic impact studies relating to the area of the condemned property might be used to cross-examine the lay witness on his knowledge of local real estate values.

Owners' opinions

The owner of the condemned property is deemed competent to give his opinion of the property's value by virtue of the knowledge that he is presumed to have as owner (*117*). Although this type of testimony is competent as a matter of law, the condemnee's natural bias has been said to derogate from the weight that a jury would otherwise accord to it. For this reason, it has been suggested that such testimony serves little more than to enable the owner to personally present his claim to the jury (*118*). Cross-examination on the basis of severance damage studies ought to both substantiate reasonable claims expressed in testimony by such an owner and delineate the true nature of any speculative claims proffered by him.

All opinion testimony on the condemned property's value must be based on the substantive rules of valuation heretofore discussed (*119*). In support of his opinion, on direct examination, the condemnation value witness should give the facts on which the valuation is based (*120*). These facts indicate the extent of the witness's familiarity with the condemned property. This familiarity naturally affects the weight that the jury will accord to the testimony. Such supporting evidence has been held indispensable to sustain the opinion (*121*). The reasons or general principles on which the opinion is based also may be given on direct examination (*122*), even though they are frequently left to be extracted on cross-examination. Severance damage study surveys and other economic impact studies should be of help by providing facts that can be relied upon by the expert opinion witness.

Supporting data

The supporting data to which the opinion witness testifies must be relevant and competent (*123*). Thus, with few exceptions, the opinion witness can testify on direct examination only to such data as would be admissible as independent evidence. Even though not admissible as independent evidence, in some jurisdictions comparable sales are admissible in support of opinion testimony on market value, as shown in appendix III. However, the hearsay rule has been somewhat relaxed in its application to the supporting data offered by expert opinion witnesses (*124*). The Oregon Supreme Court seems to have fashioned another exception to the hearsay rule; it has held that a real estate appraiser may properly introduce, as supporting evidence for his expert opinion, reports made by other investigators that he deems reliable (*125*). Other courts have indicated a similar inclination (*126*). Severance damage studies and other economic impact studies would seem to qualify under such a hearsay exception. The need for hearsay exception to allow the use of these studies as evidence is discussed in the next section of this article.

ANALOGIES TO OTHER FIELDS OF LAW

The need for improvement in means for determining fair compensation for land taken for highways has been described. The rapid changes in evidentiary practice occurring in other fields of law have not extended to the conventional rules governing the admissibility of evidence in condemnation cases involving land acquisition, particularly for highway right-of-way. Avoidance of the use of many types of economic facts in highway condemnation proceedings has been cited; particularly noted were some cases in which the courts' decisions were required as to valuations for remainders in partial takings, evaluation of benefits or damages resulting therefrom, and/or predictions of possibilities and probabilities of the effects of partial takings on remainders. Ways and means of ascertaining the answers for the problems posed, and related problems, heretofore have not been available. The difficulties engendered in obtaining the admission of economic research evidence in highway cases probably have resulted either from a lack or a shortage of economic factual data with which to make and support land valuations. A simplification and liberalization of the present exclusionary rules is needed; this would permit the use of economic research evidence and thereby allow more extensive reference to, and reliance on, data obtained by land economic studies, statistical surveys, samples, and opinion polls. Such data would serve as aids to the courts for ascertaining the economic facts relevant to the determination of land valuations.

Recognition of Research Evidence

Despite the frequent exclusion of research evidence as a device for evaluating land in condemnation proceedings, judicial recognition and acceptance of certain types of research evidence have occurred in various areas of litigation that may be applicable to eminent domain cases. Statistical data, summarized in census and other reports, and mortality and annuity tables are judicially noticed and at times have been admitted into evidence without a showing of the trustworthiness of the report or table. Various fields of commercial law, both private and public, have resorted to the findings of economic research. Market reports and price lists are admitted as evidence for the determination of the value of personal property. Authoritative works of scholarship, traffic surveys, and socio-economic data are admitted into the courts as independent evidence.

In this section, it is the intention to point the way toward the admission of similar research evidence in condemnation proceedings. "Law is a progressive thing. It is an expansive thing, adapting itself to new relations and interests of men. They are constantly springing up in the progress of society. But this progress must be by analogy to what is already settled" (*127*). Analogy, then, is the keystone of this section. If economic data can be admitted in the form of census reports and statistical tables and used as a yardstick for determining the value of personal property, these data should be usable as independent evidence and as circumstantial evidence, where necessary, on which the expert can rely in determining land valuations.

The emphasis here, therefore, is to present the state of the law and practice in the admission and use of research evidence in various types of cases, and to advocate the use of economic research evidence in condemnation proceedings.

Previously, the applicability of various kinds of evidence in condemnation proceedings was discussed. Parallel to a study of this nature are: (1) A consideration of the best methods of preparation and presentation of research evidence. (2) A formulation of standards to guide lawyers and courts in the presentation of economic research findings of various kinds. (3) An analysis, where economic research is germane, of widely differing situations and their classifications. (4) The limits on admissibility of economic research in courts of law. These various points will be referred to only indirectly as they are not the principal subject of this study.

Objectives

The doctrines of evidence, their applications, and the decisions stating them are as numerous as the sands of the sea. Therefore, the treatment here on the admissibility of research evidence is highly selective and demonstrative, with no pretensions to completeness. The objective is to furnish the appraiser and the lawyer with a starting point for improving measures of determining land values, so that the landowner whose property is taken or damaged will receive fair compensation. The admission of the results of economic research is one direction in which improvement may be made. Hence, a study of the use of economic research evidence in the areas of law where it is accepted and an understanding of its application are indispensable to those recommending its use in condemnation proceedings.

Admissibility and Use of Research Evidence

Factfinding is the pillar on which all judicial applications of law depend (*128*). Ascertaining facts is not always limited to the determination of facts and circumstances within the knowledge of a relatively small group, namely, the parties to the action and their supporting witnesses. Frequently, complex issues in dispute compel recourse to an almost boundless group from which information is collected, analyzed, and summarized in order to make generalizations, which are reflected in statistical tables or series. Whether this type of factual statistical data is hearsay (*129*) and, if so, whether necessity or practical convenience provides sufficient justification for excepting it to the tenets of the hearsay rule is a matter begging judicial decision.

Admission or refusal of such hearsay data (or the acceptance of some hearsay evidence and rejection of some) is based on judicial recognition that hearsay is not all more or less alike or amenable to being dealt with in a simple or uniform manner. There are many types of hearsay evidence (*130*); they are as numerous and as variegated as the types of communication, ranging from third-stage rumors to sworn affidavits of credible observers (*131*). Correspondingly, the trustworthiness of hearsay evidence scales from utter worthlessness to the highest reliability and depends upon the human frailties of perception, memory, and veracity. Such recognizance concedes that evidence is not taboo merely because of its hearsay nature, but that recognition of hearsay evidence and its admission depends on the court's determination of its reliability.

Statistical data accepted as evidence

Only two types of statistical or survey data are unquestionably admitted as independent evidence for the truth of the matter asserted therein (*132*). These two surveys are the U.S. Census reports, samples as well as complete enumerations (*133*), and mortality tables used in the computation of annuities, life insurance sums, dower, and damages for loss of life (*134*). In addition to their admission into evidence, they also may be, and frequently are, judicially noticed by the court (*135*), with all evidence to prove the facts contained therein being dispensed with. Even if admitted into evidence, the party proffering them usually is not required to make a preliminary showing as to their source, methods of compilation, authenticity, or reliability (*136*).

The courts have explained that census reports have a status of admissibility, which is withheld from other research reports, because of: (1) The confidence commanded by the disinterested manner in which census reports are compiled, (2) the trustworthiness and reliability of the expertness reflected therein, and (3) the impossibility of verifying data provided by interviewers because such information is of a privileged nature (*137*).

Mortality tables have been admitted on the general principle that they are founded on certain and constant data and deal with exact sciences (*138*). Such a reason seems to imply that every collection of figures that savors of the exact sciences is sufficient to be admitted; but, present day practices discredit such a notion. The more plausible reason for their admission, to the exclusion of others, is that the admission of this collection of data is demanded by custom and practical convenience and is relied upon by those members of the general public interested in such data (*138*). Consequently, the judiciary has relented to its use in the absence of a better yardstick for its problem-solving tasks (*135*).

The admissibility of standard tables or reports of scientific calculations of all sorts, as discovered in severance damage studies, economic impact studies, and other research results, in some circumstances may be argued for on the analogy of this exception for U.S. Census reports and mortality tables (*138*). Whether such a general rule can be regarded as established on the basis of the admission of such data is doubtful, but some decisions have been made that would seem to suggest this trend (*139*).

Commercial lists and reports

Recognition, as being within an exception to the hearsay rule, is also given to certain commercial and professional lists and reports namely, market reports, price lists, and quotations contained in newspapers and trade journals (*140*). "These documents may be described as privately printed documents published for the use of the trade or profession, or public generally, containing statements of contemporaneous facts that are accepted as reliable and acted upon by persons to whom they are furnished, and attaining currency solely because of the accuracy of their statements" (*141*). Their admission in some cases is premised upon judicial principle (*142*), in others, upon statutory mandate (*143*) that, in most instances, have carried out hints originally given by the courts.

Such data are deemed to be competent evidence of the state of the market and sufficient for informing courts of justice as to market value, because they are based upon a general survey of the whole market and because they constantly are being received and acted upon by persons who transact commercial operations on the faith of them (*144*). Their trustworthiness is found in the fact that these commercial lists are prepared for use in the trade or profession and are, therefore, habitually made with meticulous care and accuracy so that they will be resorted to and relied upon for business and commercial purposes. Also, the composers and writers of these reports and lists know that their work will have no commercial or professional market

value unless it is accurate, and that any inaccuracies more than likely will be discovered. Moreover, composers and writers of commercial lists have no motives to deceive the users thereof. The constant use of such reports and lists also tests their accuracy and sanctions their reliability (*145*).

Unlike census reports and mortality tables, market reports and price lists have not enjoyed the status of being universally admitted, without attached qualifications, as an exception to the hearsay rule. An appreciable number of States follow the Michigan rule (*146*), which requires some evidence to show: (1) either how the trade journal or newspaper obtains its information or (2) that those dealing in the trade or profession rely on such newspaper or journal for information as to market value. A few courts have yet to depart from the application of the strict New York rule (*147*), later modified by New York (*148*), that requires a prior showing of source and method of compilation. Such a requirement can present almost insuperable problems of proof in cases where the market value at a distant point is in issue and it becomes necessary to use documents that originated at that point, and/or when the market report covers a large region or even the whole country (*149*). Several jurisdictions consistently have admitted documentary evidence as to market value without a decision as to the necessity of a prior showing of trustworthiness; many of these decisions are accompanied by language that raises a question as to whether any such foundation was laid or was required to be laid (*150*).

No question is raised here as to whether a prior showing of trustworthiness or some substitute therefor is a proper rule for admitting such documentary evidence. It is indeed rare to regard as reversible error the failure to require a preliminary showing of source or general reliance in the introduction of documentary evidence. In many cases, such a preliminary showing has consisted only of testimony by the party offering the document (*151*). It is questionable whether such a showing constituted any greater guarantee of trustworthiness than the document itself provided. If the opposing party has equal access to price information and market data and equal opportunity to introduce evidence on a point under the adversary theory of procedure, he should not be allowed to win the point without doing more than standing up to object. In some cases, the time and money involved in the trial of a lawsuit has been ignored, and recovery on an admittedly good cause of action has been limited to nominal damages because of the failure to make a showing (*152*).

Oral testimony

As a corollary to the admission of commercial documents, it would seem that oral testimony based on such documents would be admissible. Such an inference has not been substantiated by case law. While most States permit an expert to base his testimony on such documents (*153*), it has been held by a small minority that such oral testimony was incompetent when based solely on documentary sources. This minority ruling has been based on the startingly incompatible ground that the documents themselves would not be admissible because they were not the best evidence (*154*). Seemingly, such decisions leave ample room for the use of documentary sources by experts, but prohibit the mere parroting of documents by the unqualified and place attention on the credibility of the writing itself (*155*). An overwhelming majority of decisions on the proof of market value by the use of documentary sources has involved the use of the documents themselves as evidence, and not as sources for oral testimony. Such a practice leads to the conclusion that, in practice at least, commercial and professional circles have adopted the better alternative.

The disinterest in the subject matter by those preparing them and the reliability of market reports and lists of current prices in journals and newspapers used by the trade, as well as census reports, mortality tables, or authoritative works in any field of scholarship, would seem to warrant their use in the courtroom as equal evidence of the facts contained therein (*156*). The legislators in a few States have tried to establish this tenor of equality by enacting statutes authorizing the use of such works as evidence as ". . . facts of general notoriety and interest" (*158*). The courts generally have declined to sanction a broad exception to the hearsay rule for such works (*159*).

Court Limitations on Admission of Research Opinion

The courts' admission into evidence of statistical surveys, samples, and research opinion evidence is still amazingly limited (*160*). Although the admission of census sampling and the averages and probabilities of mortality tables would seem to sanction the admission of other survey data, the courts have not so reasoned. While the substance of samples, opinion research, and other collections of data have been said to possess at least equal inductive value, being made with equal or greater thoroughness, sifted, arranged, and stated by trained observers, they are by the same discriminative authority relegated to the limbo of hearsay and other judicial abominations. The error lies not in looking too leniently upon census samples and mortality tables, but in a misconception of the true qualities of other scientific work (*161*).

Statistics are a science, the study and application of statistics require expert knowledge and method; this science is the process by which decisions are made, based upon incomplete knowledge. It also is a process used for generalizing from a part to the whole; it is used in attempts to solve a group of problems treated in philosophy by inductive logic. Statistical inferences are inductive because certain traits are assigned to large accumulations of objects by knowledge of these same traits for only a few of these objects (*162*). Like mortality tables, such statistical data also are founded in the theory of probability, which permits measurement of the magnitude of possible error in the result and a definite probability statement about the uncertainty of the inference (*163*).

Up to the present time, statistical surveys, samples, and opinion polls have been used sparingly in judicial problem solving; their use has been limited to admission as an exception to the general rule, not for the truth of the matter asserted but for the fact that it was made (*164*). Additional limitations have permitted such data to be used only in certain litigable areas. These areas comprise: Commercial law, both public and private; patent and trademark infringement; unfair competition; deceptive advertising; misbranding; and related areas (*165*) where consumer reaction is important. To a limited extent, their admission in antitrust cases has given some credence to surveys and opinion polls (*166*).

Use of statistical evidence

There are numerous areas of law in which statistical surveys and sampling of opinion (*167*) may be crucial to the disposition of a case, and in which such surveys have been used. It already has been indicated that statistical survey research is most important in cases involving commercial disagreements. Such surveys and polls also have been used in the fields of law dealing with immigration, naturalization, and deportation (*168*); in cases involving change of venue (*169*); and in quasi-legislative proceedings (*170*). On the other hand, in litigation concerned with property valuation and condemnation, surveys and polls rarely have been utilized in evidence, and these could be most useful (*171*).

Sampling results considered hearsay

Admission of sampling results has been limited in judicial proceedings because of technical objections; such results are considered to be hearsay evidence that relies on out of court statements as to the characteristics of basic data or sample data. It has been held that the conclusion of the statistician is merely opinion as to matters that do not fall within the range of admissible opinion evidence, and that statistical data and inferences therefrom are not the best evidence available of the characteristics of basic data (*172*). Evidence from sampling and polling also has been subjected to the suspicions of judges who realized the ease with which over zealous lawyers, seeking to advance the cause of their clients, could have been tempted to bias such data, and the difficulty of detecting such bias (*173*). Reputable research organizations, however, enjoy the same confidential relationship to their clients as do reputable members of the bar: They will not countenance perjured testimony in their behalf. Their system of analysis, design of experiment, and the full results of their efforts are all open to judicial review, the court willing. The legitimate opinion research organization wants its findings to be considered public property in the sense that they cannot be perverted in support of any single side and that the full implications, involving qualifications where they exist, be revealed (*174*).

Another factor militating against the admissibility of sampling and polling data as evidence is that adverse parties sometimes offer other polls by which they purport to prove inconsistent propositions of fact are shown in the data presented as evidence (*175*). However, in such instances, it would seem that the conflicting testimony on the reliability of the data should be considered in connection with the credibility of the evidence and not with the admissibility of the evidence.

Minimizing objections to sampling

To minimize the bases for objection to the admission of statistical data as evidence, the following procedures might be helpful: (1) The use of pretrial conferences, where feasible, for having the parties start with the same set of instructions and the same basic facts (*176*). (2) The service upon the adversary, in advance of trial, of a copy of the statistical report, along with a statement of the underlying materials, their location, and availability for inspection (*177*). (3) The qualifying of the official who conducted the research, by the party offering the document. (4) The testimony of the official as an authenticating witness if the adverse party requests it and shows cause (*178*).

If the hearsay objection is considered too difficult to be overcome in getting evidence of the poll or sample into the record for the court's consideration, another basis remains for bringing the results to the attention of the court. Quite often, a judge will take judicial notice of well-known facts and opinions (*179*). In this connection, statistical research findings deserve consideration by the judge as an alternative to his impressions. When used in this sense, as judicial notice, such statistical research findings need not conform to the technical rules of evidence.

REASONS FOR USE OF RESEARCH EVIDENCE IN CONDEMNATION PROCEEDINGS

The current use, and the objectives and extent of the usage, of research evidence in condemnation proceedings has been described. It seems worthwhile to summarize here the reasons considered valid for the general acceptance and use of research evidence in condemnation proceedings.

- Economic research would become an additional step in the evolutionary process of obtaining adequate and accurate ways and means of estimating the value of land. Already in the evolutionary process, comparable sales information on particular parcels is admissible in some States as direct evidence, if the foundation for each parcel is separately and individually made. As was pointed out in connection with the discussion on economic data in condemnation proceedings, a means of obtaining suitable comparable sales data and other relevant facts associated with such sales is now available. The next step of admitting the same type of sale information in aggregate or statistical form should be taken. Suppose one issue in a condemnation proceeding is the determination of the trend in land prices for a particular community over a period of years. The only way to determine precisely such prices is to tabulate records of sales, which may run into tens, hundreds, or even thousands of individual sales. In such a case, admission of survey evidence appears to be essential; it would save time and money while keeping the record clear of the various underlying source materials.

- "To preserve the vitality of its functions, the law, as it relates to the market place, must keep pace with evolutions in the market place" (*180*). Research evidence is the keystone of all of today's problem-solving methods. Its use has been pinpointed in the courtroom, as well as in commercial and professional circles.

- Analytically, the general types of land economic studies and land value surveys, which have been discussed, may be designated as hearsay because they are based upon valuations of property made by persons not represented in these proceedings. But, the principles that have supported the admission of census reports, mortality tables, market reports, and price lists will, and should, allow such economic data to be given as evidence in condemnation proceedings. From this brief specified study of research evidence and the hearsay rule in judicial proceedings, it is apparent that two main hurdles must be overcome before economic research evidence such as land economic studies and surveys will be admitted as an exception to the hearsay rule.

- The first hurdle is necessity. The information presented here shows clearly that facts on which land value estimates can be substantiated and supported are needed urgently by public officials, fee appraisers, lawyers, and juries. The courts are sufficiently aware of this need, for in 1960 alone, 16 appeals cases were handed down in which the only issue on appeal was whether the verdict was supported by the evidence (*181*). In four cases, the lower courts had made awards that shocked the respective appellate courts so that the original awards were reversed (*182*). In addition, decisions on three cases were reversed on the findings that the awards were not within the range of evidence (*183*). Acceptance of the results of: (1) a scientifically designed sample of sales prices for properties within an area, (2) a properly prepared and conducted opinion survey designed to determine various influences on land values, (3) an impact study (4) a severance damage study, and/or (5) other economic data as an exception to the hearsay rule would seem to hold the promise of furnishing material to meet this shortage of factual data.

- The second hurdle is the trustworthiness of the document. The guarantee that such economic studies and statistics would be trustworthy and reliable is to be found in the conditions and procedures with respect to their preparation. In addition, the State highway departments or the universities associated with them in these endeavors would be unlikely to stake their reputations upon ill-conceived studies. The motive, in other words, is precisely the same in character and is more certain in its influence than that which is accepted as sufficient in some of the other hearsay exceptions previously discussed; it is the unwelcome probability of a detection and exposure of errors.

- In addition to their admission as an exception to the hearsay rule, there seem to be good reasons for land economic studies and surveys to come into evidence through judicial notice. Such admission would of necessity be premised upon their undisputed authenticity, thereby obviating the necessity for testimony by a witness vouching for such.

- The public document rule also seems to be another vehicle that can be used as a reason for admitting these land studies as evidence in condemnation proceedings. Their admission under such an exception would depend upon whether they had been prepared by governmental agencies, within the scope of duty imposed upon them by law, or whether it was the usual course of business for highway departments to make certain land value studies in connection with condemnation. The important point is that the law of evidence is changing; in many fields of law it is moving in the direction of admission of factual data derived from studies and surveys, and the applications of statistical techniques.

(*References begin on p. 33*)

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(18) *Land Economic Study Committee Report to Membership*, Buckeye Chapter, American Right of Way Association, Columbus, Ohio, Sept. 1959 (unpublished).

(19) *Washington State Severance Damage Study, Nos. 7 and 9*, Washington State Highway Commission, Sept. 1959.

(20) *Preliminary Report of Land Economic Studies*, Ohio Department of Highways in cooperation with the Bureau of Public Roads, 1960.

(21) *U.S. Constitution, Amendment V*, which provides ". . . nor shall private property be taken for public use without just compensation, . . ." binds the Federal Government. The due process clause of *U.S. Constitution, Amendment XIV*, as construed in *Chicago, B. & Q. R.R.* v. *Chicago*, 166 U.S. 226 (1897) imposes substantially the same requirement on the States.

(22) *United States* v. *Causby*, 328 U.S. 256 (1946); *Portsmouth Harbor Land & Hotel Co.* v. *United States*, 260 U.S. 327 (1922); *Friendship Cemetery* v. *City of Baltimore*, 197 Md. 610, 81 A. 2d 57 (1951); *Penn* v. *Carolina Va. Corp.*, 231 N.C. 481, 57 S.E. 2d 817 (1950); *Cochran Coal Co.* v. *Municipal Management Co.*, 380 Pa. 397, 110 A. 2d 345 (1955). This traditional concept of taking has been broadened to render compensable the substantial interference with any of the rights of property ownersbip in some States. *E.g., In re Forrstrom*, 44 Ariz. 472, 38 P. 2d 878 (1934); *Liddick* v. *City of Council Bluffs*, 232 Iowa 197, 5 N.W. 2d 361 (1942); *State ex rel. McKay* v. *Kaner*, 156 Ohio St. 347, 102 N.E. 2d 703 (1951).

(23) *Chicago* v. *Taylor*, 125 U.S. 161 (1888); *Jarnagin* v. *Louisiana Highway Commission*, 5 So. 2d 660 (La. App. 1942); *Wolfrom* v. *State*, 246 Minn. 264, 74 N.W. 2d 510 (1956); *State Highway Commission* v. *Bloom*, 77 S. Dak. 452, 93 N.W. 2d 572 (1958).

(24) *Danforth* v. *United States*, 308 U.S. 271 (1939); see cases cited in reference *26*.

(25) *Rose* v. *State*, 19 Calif. 2d 713, 737–40, 123 P. 2d 505, 519–21 (1942); *State Highway Board* v. *Coleman*, 78 Ga. App. 54, 50 S.E. 2d 262 (1948); *Harrison* v. *Louisiana Highway Commission*, 191 La. 839, 186 So. 354 (1939); *Tennessee Gas Transmission Co.* v. *Maze*, 45 N.J. Super. 496, 133 A. 2d 28 App. Div. (1957).

(26) *De Bruhl* v. *State Highway & Public Works Commission*, 247 N.C. 671, 102 S.E. 2d 229 (1958); *In re Appropriation for Highway Purposes*, 167 Ohio St. 463, 150 N.E. 2d 30 (1958); *White* v. *State Highway Commission*, 201 Va. 885, 114 S.E 2d 614 (1960). But this uniformly designated valuation data varies according to the different acts that constitute a taking as illustrated by these same cases: *De Bruhl* (an administrative order); *In re Appropriation* (entry on the premises); *White* (initiation of legal proceedings).

(27) *Olson* v. *United States*, 292 U.S. 246 (1934); *Housing Authority* v. *Lustig*, 139 Conn. 73, 90 A. 2d 169 (1952); *Hoy* v. *Kansas Turnpike Authority*, 184 Kans. 70, 334 P. 2d 315 (1959); *State ex rel. Department of Highways* v. *Tolmas*, 238 La. 1, 113 So. 2d 288 (1959); *State Highway Commission* v. *Superbilt Mfg. Co.*, 204 Oreg. 393, 281 P. 2d 707 (1955).

(28) *Newton Girl Scout Council* v. *Massachusetts Turnpike Authority*, 335 Mass. 189, 138 N.E. 2d 769 (1956); *Assembly of God Church* v. *Vallone*, 150 A. 2d 11 (R.I. 1959).

(29) The market value concept seems to be synonymous with fair market value and cash market value insofar as they are adapted to eminent domain law. Orgel, *Valuation Under Eminent Domain*, §17 (1953).

(30) *Assembly of God Church* v. *Vallone*, 150 A. 2d 11, 15 (R.I. 1959). See cases cited in reference *27*.

(31) *United States* v. *Miller*, 317 U.S. 369 (1943); *State ex rel. Department of Highways* v. *Barrow*, 238 La. 887, 116 So. 2d 703 (1959); *Schlotman* v. *Wharton County*, 253 S.W. 2d 325 (Tex. Civ. App. 1953); *Pruner* v. *State Highway Commissioner*, 173 Va. 307, 4 S.E. 2d 393 (1939).

(32) *United States ex rel. T.V.A.* v. *Powelson*, 319 U.S. 266 (1943); see cases cited in references *43* and *47*.

(33) *Wilmington Housing Authority* v. *Harris*, 47 Del. 469, 93 A. 2d 518 (Super. Ct. 1952); *City of St. Louis* v. *Paramount Mfg. Co.*, 247 Mo. App. 200, 168 S.W. 2d 149 (1943); *Syracuse University* v. *Stote*, 7 Misc. 2d 349, 166 N.Y.S. 2d 402 (Sup. Ct. 1957).

(34) *Housing Authority* v. *Lustig*, 139 Conn. 73, 90 A. 2d 169 (1952); *Olson* v. *United States*, 292 U.S. 246 (1934).

(35) *Department of Public Works & Buildings* v. *Lambert*, 411 Ill. 183, 103 N.E. 2d 356 (1952); *Hoy* v. *Kansas Turnpike Authority*, 184 Kans. 70, 334 P. 2d 315 (1959).

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(37) *Newton Girl Scout Council* v. *Massachusetts Turnpike Authority*, 335 Mass. 189, 138 N.E. 2d 769 (1956) (summer camp); *Assembly of God Church* v. *Vallone*, 150 A. 2d 11 (R.I. 1959) (church).

(38) *State ex rel. Department of Highways* v. *Barrow* 238 La. 887, 116 So. 2d 703 (1959) (by stipulation) *Tigar* v. *Mystic River Bridge Authority*, 329 Mass. 514, 109 N.E. 2d 148 (1952) (partially built refrigeration plant).

(39) See references *37* and *38*.

(40) See reference *37*.

(41) *Assembly of God Church* v. *Vallone*, 150 A. 2d 11 (R.I. 1959).

(42) *Jackson* v. *State*, 213 N.Y. 34, 106 N.E. 758 (1914); *State Highway Commission* v. *Superbilt Mfg. Co.*, 204 Oreg. 393, 281 P. 2d 707 (1955).

(43) *Department of Public Works & Buildings* v. *Lambert*, 411 Ill. 483, 103 N.E. 2d 356 (1952); *In re Smith St. Bridge*, 234 App. Div. 583, 255 N.Y.S. 801 (1932) *Williams* v. *State Highway Commission*, 252 N.C. 141, 113 S.E. 2d 263 (1960); *State Highway Commission* v. *Vella*, 213 Oreg. 386, 323 P. 2d 941 (1958); *Ryan* v. *Davis*, 201 Va. 79, 109 S.E. 2d 409 (1959).

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(68) *State* v. *Stoner*, Ala., 122 So. 2d 115 (1960); *Gabriel* v. *Cox*, 130 Conn. 165, 32 A. 2d 649 (1943); *Barnes* v. *North Carolina State Highway Commission*, 250 N.C. 378, 109 S.E. 2d 219 (1959); Johnson's Petition, 344 Pa. 5, 23 A. 2d 880 (1942).

(69) *People ex rel. Department of Public Works* v. *Loop*, 127 Calif. App. 2d 786, 274 P. 2d 885 (1954), *State Highway Board* v. *Bridges*, 60 Ga. App. 240, 3 S.E. 2d 907 (1939); *Department of Public Works & Buildings* v. *Griffin*, 305 Ill. 585, 137 N.E. 523 (1922).

(70) *People ex rel. Department of Public Works* v. *Schultz Co.*, 123 Calif. App. 2d 925, 268 P. 2d 117 (1954) *State Highway Board* v. *Bridges*, 60 Ga. App. 240, 3 S.E. 2d 907 (1939); *State ex rel. State Highway Commission* v. *White*, 251 S.W. 2d 668 (Mo. App. 1953); *D'Angelo* v. *Director of Public Works*, 152 A. 2d 211 (R.I. 1959).

(71) *Department of Public Works* v. *Barton*, 371 Ill. 11, 19 N.E. 2d 935 (1939); *In re Appropriation for Highway Purposes*, 93 Ohio App. 179, 112 N.E. 2d 411 (1952); *State Highway Commission* v. *Bailey*, 212 Oreg. 261, 319 P. 2d 906 (1957).

(72) See reference 70.

(73) See references 70 and 71.

(74) *Sorensen* v. *Cox*, 132 Conn. 583, 586–87, 46 A. 2d 125, 126 (1946).

(75) Alabama: Ala. Code Ann., tit. 19, §§ 4, 10–16 (1940) (with right to appeal to common law jury in trial *de novo*) Georgia: Ga. Code Ann., § 36–401–36–403 (1933) with right to appeal to common law jury in trial *de novo*); Missouri: Mo. Rev. Stat., § 523.040 (1959)(with right to appeal to common law jury in trial *de novo*); Virginia: Va. Code, § 33–63 (1950) (without right to appeal to common law jury in trial *de novo*).

(76) Arizona: Ariz. Const. art. 2, § 23; Ariz. Rev. Stat. Ann., §§ 1146–47 (1956); Florida: Fla. Const. art. 16, § 29; Fla. Stat., § 73.10 (1957); Illinois: Ill. Const. art. 11, § 13 (not applicable to the State); Ill. Rev. Stat., c. 47, § 1 (Supp. 1960) (applicable to the State); Massachusetts: Mass. Gen. Laws Ann., c. 79, § 22, c. 80A, § 9 (1958).

(77) *Shoemaker* v. *United States*, 147 U.S. 282, 303–06 (1893); *In re Bronx Parkway Commission*, 206 App. Div. 526, 202 N.Y.S. 249 (1923). *Pruner* v. *State Highway Commissioner*, 173 Va. 307, 4 S.E. 2d 393 (1939).

(78) *Shoemaker* v. *United States*, 147 U.S. 282, 303–06 (1893).

(79) *City of Chicago* v. *Harbecke*, 409 Ill. 425, 100 N.E. 2d 616 (1951).

(80) *Hance* v. *State Roads Commission*, 221 Md. 164, 171, 156 A. 2d 644, 647 (1959).

(81) *Epstein* v. *Boston Housing Authority*, 317 Mass. 297, 299, 58 N.E. 2d 135, 137 (1944).

(82) *Epstein* v. *City & County of Denver*, 133 Colo. 104, 293 P. 2d 308 (1956); *Mississippi State Highway Commission* v. *Taylor*, 237 Miss. 847, 116 So. 2d 757 (1960); *In re Ohio Turnpike Commission*, 164 Ohio St. 377, 131 N.E. 2d 397 (1955); cert. denied 352 U.S. 806 (1957); *B & K, Inc.* v. *Commonwealth*, 398 Pa. 518, 159 A. 2d 206 (1960).

(83) *Mississippi State Highway Commission* v. *Taylor*, 237 Miss. 847, 853, 116 So. 2d 757, 760 (1960); *cf. Eames* v. *Southern N.H. Hydro-Elect. Corp.*, 85 N.H. 379, 381–82, 159 Atl. 128, 129 (1932); *State* v. *Peek*, 1 Utah 2d 263, 271, 265 P. 2d 630, 636 (1953).

(84) *Epstein* v. *City & County of Denver*, 133 Colo. 104, 108–09, 293 P. 2d 308, 310 (1956).

(85) *State* v. *Peek*, 1 Utah 2d 263, 273, 265 P. 2d 630, 637 (1953); *Templeton* v. *State Highway Commission*, 254 N.C. 337, 118 S.E. 2d 918 (1961); *Pittsburgh Terminal Warehouse & Transfer Co.* v. *Pittsburgh*, 330 Pa. 72, 198 Atl. 632 (1938).

(86) *County of Los Angeles* v. *Faus*, 48 Calif. 2d 672, 312 P. 2d 680 (1957); *Redfield* v. *Iowa State Highway Commission*, 251 Iowa 332, 99 N.W. 2d 413 (1959); *Village of Lawrence* v. *Greenwood*, 300 N.Y. 231, 90 N.E. 2d 53 (1949).

(87) *Stewart* v. *Commonwealth*, 337 S.W. 2d 880, 884 (1960); *State* v. *Peek*, 1 Utah 2d 263, 272, 265 P. 2d 630, 636 (1953).

(88) *Forest Preserve Dist.* v. *Kean*, 298 Ill. 37, 131 N.E. 117 (1921).

(89) *Stewart* v. *Commonwealth*, 337 S.W. 2d 880, 883 (1960); *State* v. *Peek*, 1 Utah 2d 263, 273, 265 P. 2d 630, 637 (1953).

(90) *County of Los Angeles* v. *Faus*, 48 Calif. 2d 672, 312 P. 2d 680 (1957); *Department of Public Works & Buildings* v. *Drabnick*, 14 Ill. 2d 28, 150 N.E. 2d 593 (1958); *Application of Port of New York Authority*, 28 N.J. Super. 575, 101 A. 2d 365 (App. Div. 1953); *State* v. *Peek*, 1 Utah 2d 263, 265 P. 2d 630 (1953).

(91) *State ex rel. Department of Highways* v. *Barber*, 238 La. 587, 115 So. 2d 864 (1959).

(92) *Gardner* v. *Brookline*, 127 Mass. 358 (1879).

(93) *Knollman* v. *United States*, 214 F. 2d 106 (6th Cir. 1954) (suitable for industrial development).

(94) *Vann* v. *State Highway Department*, 95 Ga. App. 243, 97 S.E. 2d 550 (1957); *Stewart* v. *Commonwealth*, 337 S.W. 2d 880 (Ky. 1960).

(95) *County of Los Angeles* v. *Faus*, 48 Calif. 2d 672, 312 P. 2d 680 (1957); *Application of Port of New York Authority*, 28 N.J. Super. 575, 101 A. 2d 365 (App. Div. 1953).

(96) *Epstein* v. *Boston Housing Authority*, 317 Mass. 297, 58 N.E. 2d 135 (1944) (Similar property); *State ex rel. State Highway Commission* v. *Rauscher*, 291 S.W. 2d 89 (Mo. 1956) (same property); *Application of Port of New York Authority*, 28 N.J. Super. 575, 101 A. 2d 365 (App. Div. 1953) (similar property); *Thompson* v. *State*, 319 S.W. 2d 368 (Tex. Civ. App. 1958) (same property).

(97) *Congregation of the Mission of St. Vincent de Paul* v. *Commonwealth*, 336 Mass. 357, 145 N.E. 2d 681 (1957); *Phelps* v. *State*, 157 S.W. 2d 955 (Tex. Civ. App. 1942).

(98) *Stewart* v. *Commonwealth*, 337 S.W. 2d 880 (Ky. 1960); *Robards* v. *State*, 285 S.W. 2d 247 (Tex. Civ. App. 1955). *Contra, County of Los Angeles* v. *Faus*, 48 Calif. 2d 672, 312 P. 2d 680 (1957).

(99) *Forest Preserve Dist.* v. *Kean*, 298 Ill. 37, 131 N.E. 117 (1921).

(100) *United States* v. *Katz*, 213 F. 2d 799 (1st Cir. 1954); cert. denied, 348 U.S. 857 (1954); *City & County of Denver* v. *Quick*, 108 Colo. 111, 113 P. 2d 999 (1941).

(101) *Phelps* v. *State*, 157 S.W. 2d 955 (Tex. Civ. App. 1942).

(102) *Redfield* v. *Iowa State Highway Commission*, 251 Iowa 332, 99 N.W. 2d 413 (1959); *cf. In re Ohio Turnpike Commission*, 164 Ohio St. 377, 131 N.E. 2d 397 (1955); *Contra, City & County of Denver* v. *Quick*, 108 Colo. 111, 113 P. 2d 999 (1941).

(103) *Montana Ry.* v. *Warren*, 137 U.S. 349 (1890).

(104) *People* v. *Al. G. Smith Co.*, 86 Calif. App. 2d 308, 194 P. 2d 750 (1948); *State* v. *Petersoa*, 134 Mont. 52, 323 P. 2d 617 (1958); *Application of Port of New York Authority*, 28 N.J. Super. 575, 101 A. 2d 365 (App. Div. 1953).

(105) *State ex rel. Department of Highways* v. *Hub Realty Co.*, 239 La. 154, 118 So. 2d 364 (1960); *Port of New York Authority* v. *Howell*, 59 N.J. Super. 343, 157 A. 2d 731 (Law Div. 1960).

(106) *Blount County* v. *Campbell*, 268 Ala. 548, 109 So. 2d 678 (1959); *State ex rel. State Highway Commission* v. *Devenyns*, 179 S.W. 2d 740 (Mo. App. 1944).

(107) *Shelby County* v. *Baker*, 269 Ala. 111, 110 So. 2d 896 (1959); *Department of Public Works & Buildings* v. *Pellini*, 7 Ill. 2d 367, 131 N.E. 2d 55 (1955); *Mazi* v. *Commonwealth*, 335 Mass. 101, 138 N.E. 2d 578 (1956).

(108) *State* v. *McDonald*, 88 Ariz. 1, 352 P. 2d 343 (1960); *Southwick* v. *Massachusetts Turnpike Authority*, 339 Mass. 666, 162 N.E. 2d 271 (1959); *Taney County* v. *Addington*, 304 S.W. 2d 842 (Mo. 1957); *South Carolina State Highway Department* v. *Hines*, 234, S.C. 254, 107 S.E. 2d 643 (1959).

(109) *Shelby County* v. *Baker*, 269 Ala. 111, 110 So. 2d 896 (1959); *Lazenby* v. *Arkansas State Highway Commission*, 231, Ark. 601, 331 S.W. 2d 705 (1960); *Forest Preserve Dist.* v. *Krol*, 12 Ill. 2d 139, 145 N.E. 2d 599 (1957); *State ex rel. State Highway Commission* v. *Devenyns*, 179 S.W. 2d 740 (Mo. App. 1944).

(110) *Chicago & W.I. R.R.* v. *Heidenreich*, 254 Ill. 231, 239–40, 98 N.E. 567, 571 (1912).

(111) *Eisenring* v. *Kansas Turnpike Authority*, 183 Kans. 774, 332 P. 2d 539 (1958); *Newton Girl Scout Council* v. *Massachusetts Turnpike Authority*, 335 Mass. 189, 138 N.E. 2d 769 (1956).

(112) *Application of Port of New York Authority*, 28 N.J. Super. 575, 579, 101 A. 2d 365, 367 (App. Div. 1953).

(113) *Shelby County* v. *Baker*, 269 Ala. 111, 110 So. 2d 896 (1959); *State* v. *McDonald*, 88 Ariz. 1, 352 P. 2d 343 (1960); *Taney County* v. *Addington*, 304 S.W. 2d 842 (Mo. 1957); *South Carolina State Highway Department* v. *Hines*, 234 S.C. 254, 107 S.E. 2d 643 (1959).

(114) *State* v. *McDonald*, 88 Ariz. 1, 352 P. 2d 343 (1960).

(115) *Shelby County* v. *Baker*, 269 Ala. 111, 110 So. 2d 896 (1959); *South Carolina State Highway Department* v. *Hines*, 234 S.C. 254, 107 S.E. 2d 643 (1959).

(116) *Taney County* v. *Addington*, 304 S.W. 2d 842 (Mo. 1957).

(117) *Arkansas State Highway Commission* v. *Covert*, 332 S.W. 2d 196 (Ark. 1960); *Randle* v. *Kansas Turnpike Authority*, 181 Kans. 416, 312 P. 2d 235 (1957); *Southwick* v. *Massachusetts, Turnpike Authority*, 339 Mass. 666, 162 N.E 2d 271 (1959); *Contra, Greene* v. *State Board of Public Roads*, 50 R.I. 489, 149 Atl. 596 (1930).

(118) *Besen* v. *State*, 17 Misc. 2d 119, 130, 185 N.Y.S. 2d 495, 504 (Ct. Cl. 1959).

(119) *Indianapolis & Cincinnati Traction Co.* v. *Wiles*, 174 Ind. 236, 91 N.E. 161 (1910); *Mississippi State Highway Commission* v. *Hillman*, 189 Miss. 850, 198 So. 565 (1940); *City of Houston* v. *Fisher*, 322 S.W. 2d 297 (Tex Civ. App. 1959).

(120) *Johnson's Petition* 344 Pa. 5, 23 A. 2d 880 (1942); *L'Etoile* v. *Director of Public Works*, 153 A. 2d 173 (R.I. 1959).

(121) *State Highway Commission* v. *Byars*, 221 Ark. 845, 256 S.W. 2d 738 (1953).

(122) *People* v. *Al. G. Smith Co.*, 86 Calif. App. 2d 308, 194 P. 2d 750 (1948); *Hance* v. *State Roads Commission*, 221 Md. 164, 156 A. 2d 644 (1959); *Fox-Wisconsin Theatres, Inc.* v. *City of Waukesha*, 253 Wis. 452, 34 N.W. 2d 783 (1948).

(123) *City & County of Denver* v. *Quick*, 108 Colo. 111, 113 P. 2d 999 (1941); *State ex rel. State Highway Commission* v. *Dockery*, 300 S.W. 2d 444 (Mo. 1957).

(124) *Corina Union High School Dist.* v. *Jobe*, 174 Calif. App. 2d 340, 345 P. 2d 78 (1959); *Newton Girl Scout Council* v. *Massachusetts Turnpike Authority*, 335 Mass. 189, 138, N.E. 2d 769 (1956); *Tennessee Gas Transmission Co.* v. *Maze*, 45 N.J. Super. 496, 133 A. 2d 28 (App. Div. 1957); *State Highway Commission* v. *Arnold*, 218 Oreg. 43, 341 P. 2d 1089 (1959); *City of Houston* v. *Huber*, 311 S.W. 2d 488 (Tex. Civ. App. 1958).

(125) *State Highway Commission* v. *Arnold*, 218 Oreg. 43, 341 P. 2d 1089 (1959).

(126) *Stewart* v. *Commonwealth*, 337 S.W. 2d 880, 885 (Ky. 1960); *Tennessee Gas Transmission Co.* v. *Maze*, 45 N.J. Super. 496, 504, 133 A. 2d 28, 32 (App. Div. 1957).

(127) Seminar on Protracted Cases, 23 F.R.D. 319, 449 (1959).

(128) See Note 20, Geo. Wash. L. Rev. 211 (1951).

(129) "Hearsay evidence is testimony in court or written evidence of a statement made out of court, such testimony being offered as an assertion to show the truth of matters asserted therein; and, thus, resting for its value upon the credibility of the out of court asserter." McCormick, *Evidence*, § 225 (1954).

(130) See reference 129, p. 301.

(131) See reference 129, p. 234.

(132) Zeisel, *The Uniqueness of Survey Evidence*, 45 Cornell L. Q. 322 (1959); McCormick, *Evidence* § 296.

(133) 13 U.S.C. 195 (1958).

(134) *Turcotte* v. *DeWitt*, 332 Mass. 160, 124 N.E. 2d 241 (1955). *Tranttoff* v. *Dannen Mills, Inc.*, 316 S.W. 2d 866 (Mo. App. 1958); *Continental Oil Co.* v. *Elias*, 307 P. 2d 849 (Okla. 1956).

(135) State census reports are also judicially noticed in the States of their origin, but reference here will only be made to the United States census reports.

Alabama: *Pickens County* v. *Jordan*, 239 Ala. 589, 196 So. 121 (1940).

Arizona: *Hernandez* v. *Frohmiller*, 68 Ariz. 202 P. 2d 854 (1959).

California: *People ex rel. Stoddard* v. *Williams*, 64 Calif. 87, 27 Pac. 939 (1883).

Colorado: *In re Constitutionality of Senate Bill No. 293*, 21 Colo. 38, 39 Pac. 522 (1895).

Florida: *Budget Commission* v. *Blocker*, 60 So. 2d 193 (Fla. 1952).

Georgia: *Tift* v. *Bush*, 209 Ga. 769, 75 S.E. 2d 805 (1953).

Idaho: *City of Twin Falls ex rel. Cannon* v. *Koehler*, 63 Idaho 562, 123 P. 2d 715 (1942).

Illinois: *Coal Creek Drainage Levee Dist.* v. *Sanitary Dist.*, 336 Ill. 11, 167 N.E. 807 (1929).

Indiana: *Groves* v. *Board of Commissioners*, 199 N.E. 137 (Ind. 1936).

Iowa: *State* v. *Braskamp*, 87 Iowa 588, 54 N.W. 532(1893).

Kansas: *Sparks* v. *Sparks*, 301 Ky. 576, 192 S.W. 2d 724 (1946).

Mississippi: *Ross* v. *Morrimac Veneer Co.*, 129 Miss. 693, 92 So. 823 (1922).

Missouri: *State* v. *Public Serv. Commissioners*, 334 Mo. 985, 70 S.W. 2d 52 (1934).

Montana: *Hill* v. *Rae*, 52 Mont. 348, 158 Pac. 826 (1916).

Nebraska: *Kokes* v. *State*, 55 Nebr. 691, 76 N.W. 467 (1898).

New Jersey: *Michaels* v. *Johnson*, 33 N.J. Super. 77, 109 A. 2d 452 (1954).

New York: *Taylor* v. *City of White Plains*, 206 Misc. 946, 135 N.Y.S. 2d 773 (Sup. Ct. 1954).

North Carolina: *Clark* v. *City of Greenville*, 221 N.C. 255, 20 S.E. 2d 56 (1942).

Oklahoma: *Jones* v. *Freeman*, 193 Okla. 554, 146 P. 2d 564 (1943), appeal dismissed, 322 U.S. 717 (1944).

Oregon: *Smith* v. *Jefferson*, 75 Oreg. 179, 146 Pac. 809 (1915).

Pennsylvania: *Commonwealth* v. *Walter*, 274 Pa. 553, 118 Atl. 510 (1922).

South Carolina: *Richards* v. *City of Columbia*, 227 S.C. 538, 88 S.E. 2d 683 (1955).

Texas: *L. E. Whitman & Co.* v. *Allen*, 64 S.W. 2d 1024 (Tex. Civ. App. 1933).

Virginia: *Shelton* v. *Sydnor*, 126 Va. 625, 102 S.E. 83 (1920).

Washington: *State* v. *Smith*, 149 Wash. 173, 270 Pac. 306 (1928), judgment adhered to on rehearing, 155 Wash. 173, 284 Pac. 796 (1930).

Wisconsin: *Grimm* v. *Bayfield County*, 174 Wis. 43, 182 N.W. 466 (1921).

Mortality tables:

Alabama: *Great So. Ry.* v. *Norrell*, 225 Ala. 503, 143 So. 904 (1932).

California: *Froeming* v. *Stockton Elec. Ry.*, 171 Calif. 401, 153 Pac. 712 (1915).

Connecticut: *Strakosch* v. *Connecticut Trust & Safe Deposit Co.*, 96 Conn. 471, 114 Atl., 660 (1921).

Florida: *Harvey* v. *Rhea*, 152 Fla. 817, 12 So. 2d 302 (1943).

Illinois: *Muhlke* v. *Tiedemann*, 280 Ill. 534, 177 N.E. 708 (1917).

Indiana: *Dallas & Mavis Forwarding Co.* v. *Hiddell*, 126 N.E. 2d 18, (166 Ind.) App. 113, (1955).

Kansas: *Knoche* v. *Meyer Sanitary Milk Co.*, 177 Kans. 423, 280 P. 2d 605 (1955).

Kentucky: *Morris* v. *Morris*, 293 S.W. 2d 243, 245 (Ky. 1956): "We think that we may fairly judicially note the Federal Government's preoccupation with a collection of statistics concerning all vital matters, not only mortality, but also pertaining to such subjects as agriculture, mining, cost of living, etc., and we also recognize the general acceptance by all people of the thorough and workmanlike job which has been done over a long period of years by various Federal agencies to such an extent that many wage contracts have geared the rise and fall of wages and salaries to the rise and fall of the cost of living indices. We know of no more accurate measurement."

Michigan: *Tandy* v. *Knox*, 313 Mich. 147, 20 N.W. 2d 844 (1945).

Missouri: *Selle* v. *Selle*, 337 Mo. 1234, 88 S.W. 2d 877 (1935).

Montana: *Stephens* v. *Elliott*, 36 Mont., 92, 92 Pac. 45 (1907).

New Jersey: *Berry* v. *President & Directors of the Bank of Manhattan Co.*, 133 N.J. Eq. 164 (1943).

North Dakota: *Guer* v. *Ryaden*, 74 N.W. 2d 361 (N. Dak. 1955).

Oregon: *Shelton* v. *Lowell*, 196 Oreg., 430, 249 P. 2d 958 (1952).

Washington: *McTernan* v. *Heroux*, 77 Wash. 2d 631, 269 P. 2d 815 (1954).

West Virginia: *Drake* v. *Clay Hardware & Supply Co.*, 157 S.E. 35 (W. Va. 1931).

(*136*) *Keast* v. *Santa Ysabel G. M. Co.*, 136 Calif. 256, 68 Pac. 771, 772 (1902): "The court may or may not require such preliminary proof of standard acceptance according to its judgment of the need therefor."; *Valenle* v. *Sierra Ry.*, 151 Calif. 534, 91 Pac. 481, 484 (1907): "In some courts it is said that such tables are admissible after proper preliminary proof of their authentieity and standard quality. Such proof in this ease was not made, but the general weight of authority is to the contrary, and permits the introduetion of such tables as are satisfactory to the court. Such a ruliug is founded upon the theory that the court may take judicial notice of standard tables."; *Hann* v. *Brooks*, 331 Ill. App. 535, 549, 73 N.E. 2d 624, 630 (1947): "A showing that the tables are used by reputable life insuranee companies is sufficient to establish their status as standard authorities." But see *Banks* v. *Braman*, 195 Mass. 97, 80 N.E. 799 (1907).

(*137*) See reference *132*, p. 325.

(*138*) 6 Wigmore, *Evidence*, § 1698 (3d ed. 1940).

(*139*) *Hultberg* v. *Phillippi*, 169 Kans. 610, 220 P. 2d 208 (1950) (motor-vehiele speed chart admitted); *Whalen* v. *Town Plan & Zoning Commission*, 146 Conn. 321, 150 A. 2d 312 (1959) (traffic reports showing the heaviest traffic in an area admitted without eomment as to its admissibility); *Bruner* v. *McCarthy*, 105 Utah 399, 142 P. 2d 649 (1943), in which exhibit containing a compilation of figures prepared by expert, based upon mortality aunuity tables for purpose of showing what amount of money it would be neeessary to invest at various interest rates to pay an individual specified amounts per year for 35 years, admitted. But see *Sloan* v. *Carolina Power & Light Co.*, 248 N.C. 125, 102 S.E. 2d 822 (1958) (table of Nat'l Elec. Safety Code issued by U.S. Dept. of Commeree, Bureau of Standards, exeluded).

For a diseussion of the admissibility of commercial and professional lists, see diseussion below; for a diseussion of the admissibility of interest tables, etc., see 6 Wigmore, *Evidence*, § 1642 (3d ed. 1940). See *United States* v. *Mortimer*, 118 F. 2d 266 (2d Cir.), cert. den., 314 U.S. 616 (1941), in which the court upheld, in a prosecution for using and conspiring to use the mails to defraud, the admission of a number of charts purporting to show defaults in the payment of taxes on a high proportion of certain mortgaged properties which had been prepared by a prosecution witness, an experienced public accountant, and the reliability of which was not questioned, even though the tax records were not themselves in evidence and all those who participated in their preparation did not testify; *San Francisco* v. *Superior Court of San Francisco*, 38 Calif. 2d 156, 238 P. 2d 581 (1951), in which the court issued a writ of prohibition to restrain enforcement of an order for the inspection of documents and data claimed to be the records of official proceedings conducted by the Civil Service Commission of San Francisco, which included a wage rate survey in which the commission solicited information from private employers on the written promise and agreement with each that the source of all information supplied would be held in confidence and that the wage scales and other data would not be identified except by a code known only to the commission, such survey being made necessary by the municipal employees in accord with the generally prevailing wages for like service conditions in private employment.

(*140*) 6 Wigmore, *Evidence* § 1702, 1704 (3d ed. 1940); McCormick, *Evidence* § 296 (1954); Comment, 45 Mich. L. Rev. 748 (1947); Note 39, Harv. L. Rev. 885 (1926).

Alabama: *Farm Industries Div. of Quaker Oats Co.* v. *Howell*, 39 Ala. App. 131, 95 So. 2d 808 (1957).

Arizona: *Atlantic Nat'l Bank* v. *Korrick*, 29 Ariz. 486, 242 Pac. 1009 (1926).

Arkansas: *St. Louis & S.F.R.R.* v. *Pearce*, 82 Ark. 353, 101 S.W. 760 (1907).

Colorado: *Estes* v. *Denver & R.G.R.R.*, 49 Colo. 378, 113 Pac. 1005 (1910).

Connecticut: *State* v. *Pambianchi*, 139 Conn. 543, 95 A. 2d 695 (1953).

Georgia: *Columbian Peanut Co.* v. *Pope*, 69 Ga. App. 26, 24 S.E. 2d 710 (1943).

Idaho: *State* v. *Jensen*, 47 Idaho 785, 280 Pac. 1039 (1929).

Illinois: *Nash* v. *Classen*, 163 Ill. 409, 45 N.E. 276 (1828).

Kansas: *Webbler* v. *Umback*, 125 Kans. 117, 263 Pac. 786 (1928).

Louisiana: *Friedman Iron & Supply Co.* v. *J. B. Beaird Co.*, 222 La. 627, 63 So. 2d 144 (1952).

Maine: *Washington Ice Co.* v. *Webster*, 68 Maine 463 (1878).

Maryland: *Jones* v. *Ortet*, 114 Md. 205, 78 Atl. 1030 (1910).

Michigan: *Sisson* v. *Cleveland & T. R.R.*, 14 Mich. 489 (1866).

Mississippi: *Dearborn Motors Credit Corp.* v. *Henton*, 221 Miss. 643, 74 So. 2d 739 (1954).

Missouri: *Bailey* v. *St. Louis & S.F. Ry.*, 209 S.W. 630 (Mo. App. 1927).

Nebraska: *Allender* v. *Chicago & N.W. Ry.*, 119 Nebr. 559, 230 N.W. 102 (1930).

New Jersey: *State* v. *Carrano*, 27 N.J. Super. 382, 99 A. 2d 426 (1953) (criminal case recognizing the rule).

New Mexico: *Johnson* v. *Nichols*, 66 N. Mex. 181, 344 P. 2d 697 (1959).

New York: *Whelan* v. *Lynch*, 60 N.Y. 469 (1875); *Watts* v. *Phillips-Jones Corp.*, 211 App. Div. 523, 207 N.Y. S. 493 (1925), Aff'd, 242 N.Y. 557, 152 N.E. 425 (1926).

North Carolina: *Commander* v. *Smith*, 192 N.C. 159, 134 S.E. 412 (1926).

North Dakota: *Schnitz Bros.* v. *Bolles & Royers Co.*, 48 N. Dak. 673, 186 N.W. 96 (1922).

Pennsylvania: *Bounomo* v. *United Distiller's Co.*, 77 Pa. Super. 113 (1921).

Rhode Island: *National Cash Register Co.*, v. *Underwood*, 56 R.I. 379, 185 Atl. 909 (1936), which recognized the rule but held that price list prepared and extended by company for exclusive reference by its salesmen, and not in any way to be used as a price quotation to the public for actual sale, was not probative evidence of value of that commodity in an open competitive market.

South Carolina: *Kirkpatrick* v. *Hardeman*, 123 S.C. 21,115 S.E. 905 (1923).

Texas: *Houston Packing Co.* v. *Spivey*, 333 S.W. 2d 423 (Tex. 1960). *Allen* v. *Payne*, 334 S.W. 2d 607 (Tex. Civ. App. 1960).

Utah: *Baglin* v. *Earl-Eagle Mining Co.*, 54 Utah 572, 184 Pac. 190 (1919).

Washington: *Cron & Dehn, Inc.* v. *Chelan Packing Co.*, 258 Wash. 167, 290 Pac. 999 (1930).

Wyoming: *Atlantic Nat'l Bank* v. *Korrick*, 29 Wyo. 468, 242 Pac. 1009 (1926).

Contra, Massachusetts: *Doherty* v. *Harris*, 230 Mass. 341, 119 N.E. 863 (1918).

(*141*) Note 39, Harv. L. Rev. 885 (1926).

(*142*) See reference *140* and 6 Wigmore, *Evidence*, § 1702 (3d ed. 1940).

(*143*) Code of Ala. ch. 7, 385 (1958); Ky. Rev. Stat. ch 355, § 2 724 (1960); Mass. Gen. Laws Ann. ch. 106, § 2 724 (1958) (but see Code Comment at the end of section; 6 Wigmore, *Evidence*, § 1704 (3d ed. 1940)). For a statement of Massachusetts law see *Doherty* v. *Harris*, 230 Mass. 341, 119 N.E. 863 (1918); N. Dak. Century Code ch. 32, § 25-04 (1960); Pa. Stat. ch. 12 A, § 2-724 (1954).

(*144*) *Sisson* v. *Cleveland & T. R.R.*, 14 Mich. 489 (1866).

(*145*) 6 Wigmore, *Evidence*, §§ 1702, 1704 (3d ed. 1940).

(*146*) For a statement of the rule, see reference *144*, Note 21, P. 496. This approach was formulated best in *Mount Vernon Brewing Co.*, v *Teschner*, 108 Md. 158, 69 Atl. 502 (1908); accord, *Fairley* v. *Smith*, 87 N.C. 367 (1882). Instead of giving an option as permitted by the Michigan rule, some courts require a showing that the document is relied upon by the trade dealing in the particular article or commodity in question. See, *Johnson* v. *Nichols*, 66 N. Mex. 881, 344 P. 2d 697 (1959). See generally, 45 Mich. L. Rev. 748 (1947); see reference *145*.

(*147*) The New York rule originated in *Whelan* v. *Lynch*, 60 N.Y. 469, 474 (1875). It is followed in *Fishel* v. *F. M. Ball & Co.*, 83 Calif. App. 128, 256 Pac. 493 (1927); *Willard* v *Mellor*, 19 Colo. 534, 36 Pac. 148 (1894); *Fountain* v. *Wabash Ry.*, 114 Mo. App. 676, 90 S.W. 393 (1905); *Schnitz Bros.* v. *Bolles & Rogers Co.*, 48 N. Dak. 673, 186 N. W. 96 (1921); *Baglin* v. *Earl-Eagle Mining Co.*, 54 Utah 572, 184 Pac. 190 (1919).

(*148*) In *Burns Mfg. Co.* v. *Clinchfield Products Corp.*, 189 App. Div. 569, 178 N.Y.S. 483 (1919), the court adopted a test of general reliance without commenting on *Whelan*. In *Watts* v. *Phillips-Jones Corp.*, 211 App. Div. 523, 207 N.Y.S. 493 (1925), the court also applied the test of general reliance, and modified *Whelan* by stating that a showing of source and method of compilation was not the only basis for qualifying a document. In *von Rectzenstein* v. *Tomlinson*, 249 N.Y. 60, 162 N.E. 584 (1928), the court expressed a preference for the test of general reliance.

(*149*) *Chicago, B. & Q. Ry.* v. *Todd*, 74 Nebr. 712, 105 N.W. 83 (1905); *Mount Vernon Brewing Co.* v. *Teschner*, 108 Md. 158, 69 A. 502 (1908); *Marden, Orth & Hastings Corp.* v. *Trans-Pacific Corp.*, 109 Wash. 296, 186 Pac. 884 (1920).

(*150*) *Webbler* v. *Umback*, 125 Kans. 117, 263 Pac. 786 (1928); *Jordan* v. *Miller*, 232 Mich. 8, 204 N.W. 708 (1925).

(*151*) *St. Louis I. M. & S. R.R.* v. *Laser*, 120 Ark. 119, 179 S.W. 189 (1915).

(*152*) *Kentucky Refining Co.* v. *Conner*, 145 Ala. 664, 39 So. 728 (1905); *Schnitz Bros.* v. *Bolles & Rogers Co.*, 48 N. Dak. 637, 186 N.W. 96 (1922).

(*153*) *Howell* v. *Hines*, 298 Mo. 282, 249 S.W. 924 (1923); *Fountain* v. *Wabash Ry.*, 114 Mo App. 676, 90 S.W. 393 (1905).

(*154*) *Doherty* v. *Harris*, 230 Mass. 341, 119 N.E. 863 (1819); *National Bank of Commerce* v. *New Bedford*, 175 Mass. 257, 56 N.E. 288 (1900).

(*155*) 45 Mich. L. Rev. 748, 752 (1947).

(*156*) 6 Wigmore, *Evidence*, §§ 1690-92 (3d ed. 1940); Note 19, St. Louis L. Rev. 353 (1934).

(*157*) California was the first State to enact such a statute: "Historical works, books of science or art, and published maps or charts, when made by persons indifferent between the parties are *prima facie* evidence of facts of general notoriety and interest." Calif. Code Civil Proced. § 1936. Other States have enacted statutes similar to the California statue: Ala Code Ann. ch. 7, § 413 (1940); Idaho Code § 9-402 (1948); Iowa Code Ann. § 622-23 (1958); Mont. Rev. Code Ann. § 93-1101-8 (1947); Nebr. Rev. Stat. § 25-1218 (1956); Oreg. Rev. Stat. § 41.670 (Supp. 1959); Utah Code Ann. § 78-25-6 (1953).

(*158*) *The Uniform Rules of Evidence*, Rule 63 (31), adapted from the *Model Code of Evidence*, Rule 529.

(*159*) Alabama is the only jurisdiction that has construed such a statute as permitting the direct admission of medical books, extracts, and treatises, without qualification as to purpose or ease. The other States having such statutes have uniformly construed these statutes as not to allow direct admission of medical works. *City of Dothan* v. *Hardy*, 237 Ala. 603, 188 So. 264 (1934), admitting such works, and the following, which deny such admission: *Brown* v. *L. A. Transit Lines*, 282 P. 2d 1032 (Calif. App. 1955); *Wilcox* v. *Crumpton*, 219 Iowa 389, 258 N.W. 704 (1935), recogniziug the rule; *Osborn* v. *Gray*, 28 Idaho 89, 152 Pac. 473 (1915). States not having such statutes follow the commou law rule

(*Continued on p. 38*)

STATE LEGAL MAXIMUM LIMITS OF MOTOR VEHICLE

Prepared by the Bureau

Line	State	Width inches[1]	Height ft.-in.	Length–feet[2]				Number of towed units[3]			Axle load–pounds				Type of r
				Single unit		Truck tractor semi-trailer	Other combination	Semi-trailer	Full trailer	Semi-trailer and full trailer	Single		Tandem		
				Truck	Bus						Statutory limit	Including statutory enforcement tolerance	Statutory limit	Including statutory enforcement tolerance	
1	Alabama	96	13-6	35	40	50	NP	1	NP	NP	18,000	19,800	36,000	39,600	Table
2	Alaska	96	12-6	35	[19] 40	60	60	1	1	2	18,000		32,000		Table-tire
3	Arizona	96	13-6	40	40	65	65	1	1	2	18,000		32,000		Table
4	Arkansas	96	13-6	35	40	50	50	1	1	NP	18,000	[7] 18,500	32,000	32,500	Spec. maxi
5	California	96	13-6	35	[9] 35	60	65	NR	NR	NR	18,000		32,000		Table
6	Colorado	[11] 96	[12] 13-6	35	40	60	[10] 60	1	2	2	18,000		36,000		Formula-sp
7	Connecticut	102	12-6	50	50	50	NP	1	NP	NP	22,400	22,848	36,000	36,720	Spec. lim.-
8	Delaware	96	[6] 12-6	40	42	50	60	1	1	2	20,000		36,000		Table-spec
9	District of Columbia	96	12-6	40	40	50	80	1	1	NP	22,000		38,000		Table
10	Florida	96	13-6	[14] 35	40	55	55	1	1	NP	20,000	22,000	40,000	44,000	Table
11	Georgia	96	13-6	[15] +39	[15] +45	50	50	1	1	NP	18,000	20,340	36,000	40,680	Spec. maxi
12	Hawaii	108	13-0	40	40	55	65	1	1	2	24,000		32,000		Formula[17]
13	Idaho	[11] 96	14-0	35	[19] 40	60	65	1	1	2	[20] 18,000		[20] 32,000		Table[20]
14	Illinois	96	13-6	42	42	[25] 55	[25] 60	1	1	2	[21] 18,000		32,000		Spec. lim.-
15	Indiana	96	13-6	36	40	50	50	1	1	2	[23] 18,000	[23] 19,000	[23] 32,000	[23] 33,000	Spec. lim.-
16	Iowa	96	13-6	35	[19] 40	50	50	1	[24] 1	NP	18,000	18,540	32,000	32,960	Table
17	Kansas	96	13-6	35	[19] 40	50	50	1	1	NP	18,000		32,000		Table
18	Kentucky	96	[12] 13-6	[26] 35	[26] 35	[27] 50	NP	1	NP	NP	18,000	[28] 18,900	32,000	[28] 33,600	Spec. lim.-
19	Louisiana	96	13-6	35	[19] 40	50	60	1	1	NP	18,000		32,000		Axle lim.-t
20	Maine	96	[30] 12-6	55	55	55	55	1	1	NP	[30] 22,000		[30] 32,000		Table-tire
21	Maryland	[18] 96	[6] 12-6	55	55	55	[40] 55	NR	NR	NR	22,400		[31] 40,000		Formula
22	Massachusetts	96	NR	35	[19] 40	50	NP	1	NP	NP	22,400		36,000		Table-spe
23	Michigan	96	13-6	35	40	55	55	1	1	2	[33] 18,000		[34] 32,000		Axle lim.
24	Minnesota	96	13-6	40	40	50	50	1	1	NP	18,000		32,000		Table
25	Mississippi	96	[6] 12-6	35	40	50	50	1	1	NP	18,000		28,650	[35] 32,000	Table-tire
26	Missouri	96	12-6	35	40	50	50	1	1	2	18,000		32,000		Table
27	Montana	[11] 96	13-6	35	40	60	60	1	1	[37] 2	18,000		32,000		Table
28	Nebraska	96	13-6	40	40	60	60	1	1	2	18,000	18,900	32,000	33,600	Table
29	Nevada	96	NR	NR	NR	NR	NR	NR	NR	NR	18,000	18,900	32,000	33,600	Table
30	New Hampshire	96	13-6	35	[35] 40	50	50	NR	NR	NR	22,400		36,000		Tables-s
31	New Jersey	[44] 96	[44] 13-6	35	[39] 35	50	[40] 50	1	1	NP	22,400	23,520	32,000	33,600	Spec. lim
32	New Mexico	[41] 96	13-6	40	40	65	65	1	1	2	21,600		34,320		Table
33	New York	96	13-0	35	[42] 35	50	50	1	1	NP	22,400		36,000		Formulo
34	North Carolina	96	[6] 12-6	35	[19] 40	[43] 50	[43] 55	1	1	NP	18,000	19,000	36,000	38,000	Spec. lim
35	North Dakota	96	13-6	[14] 35	[19] 40	60	60	1	1	2	18,000		32,000		Formula
36	Ohio	96	13-6	35	[19] 40	50	60	1	NR	NR	19,000		31,500		Formula
37	Oklahoma	96	13-6	35	45	[45] 60	[45] 60	1	1	NP	18,000		32,000		Table
37	Oregon	96	[12] 13-6	35	[35] 40	[46,35] 55	[35] 65	1	1	[35] 2	[47] 18,000		[47] 32,000		Table[48]
38	Pennsylvania	96	[6] 12-6	35	[19] 40	[56] 50	[40] 50	1	1	NP	22,400	23,072	36,000	37,080	Spec. lim
39	Puerto Rico	96	12-6	35	35	50	50	1	1	NP	N5		N5		Spec. lim
40	Rhode Island	102	12-6	40	40	50	50	1	1	NP	22,400		N5		Spec. lim
42	South Carolina	96	13-6	[14] 35	[19] 40	55	[59] 60	1	1	NP	20,000		32,000		Table
43	South Dakota	96	13-6	35	40	60	60	1	1	2	18,000		32,000		Table
44	Tennessee	96	[6] 12-6	35	40	50	50	1	[53] 1	NP	18,000		32,000		Table
45	Texas	96	13-6	35	40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Table
46	Utah	96	14-0	45	45	60	60	NR	NR	NR	18,000		[64] 33,000		Table[65]
47	Vermont	96	12-6	50	50	50	50	1	1	NP	N5		N5		Spec. lim
48	Virginia	96	[6] 12-6	35	[35] 40	50	50	1	1	NP	18,000		[57] 32,000		Table
49	Washington	96	13-6	35	[19] 40	60	[58] 65	1	1	[58] 2	18,000	18,000	32,000	32,000	Table-sp
50	West Virginia	96	[6] 12-6	35	[19] 40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Table
51	Wisconsin	96	13-6	35	40	50	50	1	1	NP	18,000	[60] 19,500	30,400	32,000	Table[61]
52	Wyoming	96	13-6	40	40	65	65	1	1	2	18,000		32,000	[62] 36,000	Table
	AASHO Policy	96	12-6	35	[19] 40	50	60	1	1	NP	18,000		32,000		Table
Number of States	Higher	3	45	18	31	23	11	5	6	25	31		30		Formula
	Same	49	7	34	16	29	9	47	42	27	21		21		Table
	Lower	0	0	0	5	0	32	0	4	0	0		1		Specific

NP—Not permitted. NR—Not restricted. N5—Not specified.

[1] Various exceptions for farm and construction equipment; public utility vehicles; house trailers; urban, suburban, and school buses; haulage of agricultural and forest products; at wheels of vehicles; for safety accessories, on designated highways, and as administratively authorized.

[2] Various exceptions for utility vehicles and loads, house trailers and mobile homes.

[3] When not specified, limited to number possible in practical combinations within permitted length limits; various exceptions for farm tractors, mobile homes, etc.

[4] Legally specified or established by administrative regulation.

[5] Computed under the following conditions to permit comparison on a uniform basis between States with different types of regulation:

A. Front axle load of 8,000 pounds.

B. Maximum practical wheelbase within applicable length limits:

(1) Minimum front overhang of 3 feet.

(2) In the case of a 4-axle truck-tractor semitrailer, rear overhang computed as necessary to distribute the maximum possible uniform load on the maximum permitted length of semitrailer to the single drive-axle of the tractor and to the tandem axles of the semitrailer, within the permitted load limits of each.

(3) In the case of a combination having 5 or more axles, minimum possible combined front and rear overhang assumed to be 5 feet, with maximum practical load on maximum permitted length of semitrailer, subject to control of loading on axle groups and on total wheelbase as applicable.

C. Including statutory enforcement tolerances as applicable.

[6] Auto transports 13 feet 6 inches; Maryland also allows 13 feet 6 inches for vehicles loaded with hay or straw, or carrying flat glass.

[7] Does not apply to combinations of adjacent load-carrying single axles.

[8] 56,000 pounds on load-carrying axles, exclusive of steering-axle load.

[9] On specific routes in urban or suburban service under special permit from P.U.C. 40 feet, also 3-axle buses with turning radius less than 45 feet without restriction.

[10] Except 3-unit combinations may use up to 65 ft. combinations on certain highways designated by the Department of Highways.

[11] Buses 102 inches on highways of surfaced width at least 20 feet or otherwise as administratively authorized.

[12] On class AA, or designated highways, 12 ft. 6 in. on other highways; log and lumber trucks limited to 12 ft. 6 in. on all highways in Oregon.

[13] Legal limit 60,000 pounds, axle spacing 27 feet or more

[14] Three-axle vehicles 40 feet.

[15] Truck 39.55 feet; bus 45.20 feet.

[16] 63,280 pounds maximum, except on roads under

[17] 700 (L+40) when L is 18' or less; 800 (L+40) w with span of 20' or over.

[18] Vehicles loaded with tobacco hogsheads—103 inc

[19] Less than three axles 35 feet.

[20] Special limits for vehicles hauling timber and tin including livestock; single axle 18,900 pounds, tandem mitted 66,000 pounds maximum at 21-foot axle spacing. foot axle spacing.

[21] On designated highways; 16,000 pounds on other

[22] Without tandem axles 45,000 pounds.

[23] On designated highways; single axle 22,400 pou excesses of weight under one or more limitations of ax front or steering axle.

[24] Towing agent must be registered for gross weigh

[25] Auto transports only, 60 feet.

[26] On designated highways; trucks 26.5 feet and bu

[27] Class AA highways; 45 feet on other highways.

[28] Class AA highways only.

[29] Maximum gross weight on Class A highways 42,0

[30] Including load 14 feet; various exceptions for ve

[31] Tandem axles spaced less than 48 inches apart

[32] Subject to axle and tabular limits.

[33] Single axle spaced less than 9 feet from nearest

[34] On designated highways only and limited to one

[35] On designated highways only.

[36] Administrative regulation—32,000 pounds allowe 2 and 5 is 28 ft. or more.

[37] Semitrailer and semitrailer converted to full trai

[38] Dual-drive axles; otherwise 40,000 pounds.

[39] Or as prescribed by P.U.C.

[40] Exception for poles, pillings, structural units,

ND WEIGHTS COMPARED WITH AASHO STANDARDS

s, December 31, 1961

mit		Specified maximum gross weight—pounds[4]						Practical maximum gross weight—pounds[5]						Line
pplicable to:		Truck		Truck-tractor semitrailer			Other combination	Truck		Truck-tractor semitrailer			Other combination	
y up s	Total wheel base only	2-axle	3-axle	3-axle	4-axle	5-axle		2-axle	3-axle	3-axle	4-axle	5-axle		
	X							27,800	47,600	47,600	60,010	64,650	NP	1
18′	Over 18′	36,000	50,000	50,000	72,000	76,800	76,800	26,000	40,000	44,000	58,000	72,000	76,800	2
18′	Over 18′							26,000	40,000	44,000	58,000	72,000	76,800	3
18′	Over 18′							26,500	40,500	45,000	59,000	65,000	65,000	4
								26,000	40,000	44,000	58,000	72,000	76,000	5
	X	30,000	46,000					26,000	44,000	44,000	62,000	76,000	76,000	6
		32,000	50,000	50,000	60,000	60,000	NP	30,848	44,720	51,000	61,200	61,200	NP	7
X		30,000	46,000	48,000	60,000	60,000	60,000	28,000	48,000	48,000	56,350	60,000	60,000	8
X								30,000	46,000	52,000	[54] 58,450	[54] 61,490	[54] 64,650	9
	X							30,000	52,000	52,000	65,200	73,095	73,095	10
							63,280	28,340	48,680	48,680	63,280	63,280	63,280	11
X								32,000	38,800	56,000	64,000	72,000	80,000	12
X								26,000	40,000	44,000	58,000	73, 280	76,800	13
		36,000	[22] 41,000	45,000	59,000	72,000	72,000	26,000	40,000	44,000	58,000	72,000	72,000	14
							72,000	27,000	41,000	45,000	59,000	[23] 73,000	[23] 73,000	15
X								26,540	40,960	45,080	59,500	73,280	NP	16
X								26,000	40,000	44,000	55,470	73, 280	73, 280	17
		36,000	50,000	54,000	59,640	73,280	NP	26,900	41,600	45,800	59,640	73,280	NP	18
								26,000	40,000	44,000	58,000	72,000	76,000	19
X		32,000	[30] 51, 800	51, 800	60, 050	70, 550	70, 550	30,000	40,000	51, 800	62, 040	70, 550	70, 550	20
	X			65,000	65,000	65,000	65,000	30,400	48,000	52,800	65,000	65,000	65,000	21
		[32] 46,000	[32] 73, 000	[32] 73, 000	[32] 73, 000	[32] 73, 000	NP	30,400	53,500	64, 300	73, 000	73, 000	NP	22
								26,000	[35] 40,000	44,000	[35] 58,000	[35] 66.000	[35] 102,000	23
X						[36] 73, 280		26,000	40,000	44,000	58,000	[36] 72, 000	72,500	24
X								26,000	[35] 40,000	44,000	59,000	[35] 64,650	[35] 64,650	25
X								26,000	40,000	44,000	55,470	64,650	64,650	26
18′	Over 18′							26,000	40,000	44,000	58,000	72,000	76,000	27
X		36,000	54,000	54,000	71,146	71,146	71,146	26,780	41,200	45,320	59,740	73,280	73,280	28
18′	Over 18′							26,900	41,600	45,800	60,500	75,200	76,800	29
	X	33,400	[38] 47,500	52,800	66,400			30,400	44,000	52,800	66,400	66,400	66,400	30
		30,000	40,000	60,000	60,000	60,000	60,000	31,500	41,600	55,040	63,000	63,000	63,000	31
18′	Over 18′							29,600	42,320	51,200	63,920	76,640	86,400	32
	X					65,000	65,000	30,400	44,000	52,800	65,000	65,000	65,000	33
		31,500	46,200	46,200	65,100	65,100	65,100	27,000	46,000	46,000	65,100	65,100	65,100	34
18′	Over 18′							26,000	38,000	44,000	56,000	[44] 64,000	[44] 64,000	35
	X							27,000	39,500	46,000	58,500	71,000	78,000	36
X								26,000	40,000	44,000	58,000	72,000	73,280	37
8′	Over 18′					[48] 76, 000	[48] 76,000	26,000	40,000	44,000	58,000	72,000	[48] 76,000	38
		33,000	47,000	50,000	60,000	60,000	62,000	31,072	45,080	51,500	61,800	61,800	63,860	39
														40
		[50] 36,000	[51] 44,000	[52] 50,000	[13] 60,000	60,000	88,000	30,400	44,000	50,000	60,000	60,000	88,000	41
X								28,000	40,000	48,000	60,000	66,839	71,115	42
X								26,000	40,000	44,000	58,000	72,000	73,280	43
	X							26,000	40,000	44,000	58,000	61,580	43,500	44
X								26,900	41,600	45,800	60,500	75,200	75,600	45
X								26,000	41,000	44,000	59,000	74,000	79,900	46
		[55a] 32, 000	[55b] 55, 000	[55c] 52, 800	[55d] 66, 400	[55d] 66, 400	[55d] 66, 400	[55a] 32, 000	[55b] 55,000	[55c] 52,800	[55d] 66, 400	[55d] 66, 400	[55d] 66, 400	47
X						[35] 56,800	[35] 56,800	26,000	40,000	44,000	56,800	56,800	56,800	48
8′	Over 18′	28,000	36,000	46,000	60,000	68,000	72,000	26,000	36,000	44,000	60,000	68,000	72,000	49
X						[66] 70, 000	[66] 70, 000	26,900	41,600	45,800	57,844	63,840	63,840	50
X								27,500	40,000	47,000	59,500	73,000	73,000	51
X							[66] 70, 000	26,000	44,000	44,000	62,000	73,950	73,950	52
X								26,000	40,000	44,000	55,470	61,490	71,900	
20	18							29	27	29	49	46	27	
								22	20	22	2	1	0	
								0	4	0	0	4	24	

y 56,000 pounds maximum.
18′; 900 (L+40) on highways having no structures

concentrates, aggregates, and agricultural products
gross weight table: vehicle with 3 or 4 axles per-
are axles permitted 79,000 pounds maximum at 43-

000 pounds; tolerance of 1,000 pounds on total of all
ight; depending upon the placing of 9000# on the

except agricultural commodities

highways.

B highways 30,000 pounds.
products and construction materials.
imitation of 36,000 pounds.

0 pounds.
notion; otherwise 26,000 pounds.

dum axles provided the distance between axles

ly.

nitted 70 feet

[41] On designated highways 102 inches.
[42] Trackless trolleys and buses 7 passengers or more, P.S.C. certificate 40 feet.
[43] Including front and rear bumpers.
[44] Vehicles in excess may be operated under special permit obtained in advance from the Department of Motor Vehicles.
[45] Auto transports only, by special permit only.
[46] 60 feet allowed truck tractor semitrailer on designated major routes.
[47] Logging vehicles permitted 7-foot wheelbase tolerance, 19,000-pound single axle, 34,000-pound tandem axle.
[48] Governs gross weight permitted on highways designated by resolution of State highway commission or by permit, otherwise 73,280.
[49] Single unit truck with 4 axles permitted 60,000 pounds.
[50] Axles spaced less than 6 feet 32,000 pounds; less than 12 feet 36,000 pounds; 12 feet or more gross weight governed by axle limit.
[51] Single vehicle with 3 or more axles spaced less than 16 feet 40,000 pounds; less than 20 feet 44,000 pounds; 20 feet or more governed by axle limit.
[52] Tractor semitrailer with 3 or more axles spaced less than 22 feet 46,000 pounds; not less than 27 feet 50,000 pounds.
[53] Limited to 3,500 pounds.
[54] Pavements only, maximum legal load for bridges 56,800 pounds.
[55] On Interstate Routes: a. 30,000 lbs.; b. 40,000 lbs.; c. 50,000 lbs.; d. 60,000 lbs
[56] Where truck-tractor is properly registered in Pennsylvania, 55 feet.
[57] Vehicles registered before July 1, 1956, permitted limits in effect January 1, 1956, for life of vehicle.
[58] Three-unit combinations and full truck and full trailer combinations on designated highways.
[59] House trailers only.
[60] Axle load 21,000 pounds on 2-axle trucks hauling peeled or unpeeled forest products cut crosswise or transporting milk from farm to market but not over Interstate system.
[61] On Class A highways. All axles of a vehicle or combination—73,000 pounds maximum. Wheel, axle, axle group and gross vehicle weights on Class B highways are 60% of weights authorized for Class A highways.
[62] Based on ruling of Attorney General.
[63] Weight limits to be established by administrative regulations
[64] For axle spacing under 8 feet.
[65] Weights are established on axle spacing of the extreme of any group.
[66] Only on certain highways, or portions thereof, designated by State Roads Commissioner, and consistent with Congressional action.

(*References continued from p. 35*)

prohibiting the use of medical works as direct evidence in the courtroom, except in certain specified cases authorized by statutes. S. C. Code § 26-142 (1952); Mass. Gen. Laws Ann. ch. 233 § 79C (1958); and Nev. Rev. Stat. § 51.040 (1960).

(*160*) See reference *132*; Sprowls, *The Admission of Sample Data into a Court of Law. A Case History*, 4 U.C.L.A. L. Rev. 222 (1957); McCoid, *The Admission of Sample Data into a Court of Law: Some Further Thoughts*, 4 U.C.L.A. L. Rev. 233 (1957); Note, *Public Opinion Surveys As Evidence*, 66 Harv. L. Rev. 498 (1953); Note, *Admissibility of Public Opinion Polls*, 37 Minn. L. Rev. 385 (1953)

(*161*) 6 Wigmore, *Evidence*, § 1698 (3d ed. 1940).

(*162*) McCoid, reference *160*, Note 37, pp. 223–24. Interested readers are referred to the following publicationsfor detailed studies on survey and poll methodology: Parten, *Surveys, Polls and Public Opinion* (1949); Cantril, *Gauging Public Opinion* (1947); Blankenship, *Consumer and Opinion Research* (1934); see the reference guide of Smith, Lasswell, and Casey, *Propaganda, Communication and Public Opinion* (1946). For a discussion of the courts' attitude toward the methodology of the taking of surveys or public opinion polls, see Annot., 76 A.L.R. 2d 619, 633–40.

(*163*) McCoid, reference *160*.

(*164*) *United States* v. *88 Cases*, 187 F. 2d 967 (3d Cir.), cert. den. 342 U.S. 861 (1951); *Hermann* v. *Newark Morning Ledger Co.*, 48 N.J. Super. 420, 138 A. 2d 61 (1958). See Zeisel's discussion, reference *160*, Note 6; 66 Harv. L. Rev., reference *160*.

(*165*) *Gulf Oil Corp.* v *F. T. C.*, 150 F. 2d 106 (5th Cir. 1945); Sorensen & Sorensen, *Responding to Objections Against the Use of Opinion-Survey Findings in the Courts*, 2 J. Marketing 133, 134 (1955); see generally Barksdale, *Use of Survey Research Findings as Legal Evidence* (1957); Caughey, *The Use of Public Polls, Surveys and Sampling as Evidence in Litigation and Particularly Trademark and Unfair Competition Cases*, 44 Calif. L. Rev. 539 (1956); Hall, *Evidence-Hearsay-Admissibility of Public Surveys in Unfair Competition Cases*, 46 Trademark Rep. 154 (1956); Kecker, *Admission in Courts of Law of Economic Data Based on Samples*, 28 J. Bus. 118 (1955); Note 20, Geo. Wash. L. Rev. 211 (1951); Note 66, Harv. L. Rev. 498 (1953); Annot., 76 A.L.R. 2d 619 (1961).

(*166*) *United States* v. *United Shoe Mach. Corp.*, 93 F. Supp. 190 (D. Mass. 1950); *United States* v. *J. I. Case Co.*, 101 F. Supp. 856 (D. Minn. 1951); but see *United States* v. *E. I. Dupont de Nemours & Co.*, 177 F. Supp. 1 (D. Ill. 1959).

(*167*) *RKO Radio Pictures* v. *Jarrico*, 128 Calif. App. 2d 172, 274 P. 2d 928, cert. denied, 349 U.S. 928 (1954); *Las Vegas Sun, Inc.* v. *Franklin*, 74 Nev. 282, 329 P. 2d 867 (1958); *Great Atlantic & Pacific Tea Co.* v. *A. & P. Trucking Corp.*, 51 N. J. Super. 412 144 A. 2d 172 (1958), modified on other grounds, 29 N. J. 455, 149 A. 2d 595 (1959). Dean, *Sampling to Produce Evidence on Which the Courts will Rely*, Current Bus. Studies No. 19, p. 6 (1954).

(*168*) In *Repouille* v. *United States*, 165 F. 2d 152, 153 (2d Cir. 1947), Judge Learned Hand stated that the courts have no Gallup poll to aid them in discovering the meaning of the "good moral character," required of any applicant for naturalization; a poll is a possible method for verifying a position as to moral justifiability of an act performed by an applicant for naturalization.

(*169*) Survey methods may be used to discover whether there is sufficient local prejudice to justify a change of venue in criminal cases. See Note 54, Harv. L. Rev. 679, 684 (1941); Sorensen, *The Role of Public Sentiment and Personal Prejudice in Jury Trials of Criminal Cases*, Ch. X (unpublished dissertation, the University of Chicago).

(*170*) Woodward, *A Scientific Attempt to Provide Evidence for a Decision on Change of Venue*, 17 Am. Sociol. Rev. 447 (1952).

(*171*) "Value is nothing more than the price for which property may be sold and the value of other like property is highly probative as to the value of the property in question . . . . In the commercial field there is no more commonly accepted method for ascertaining property values than by comparison with other property and the prices at which it is sold." *City of Los Angeles* v. *Cole*, 28 Calif. 2d 509, 521, 170 P. 2d 928, 934 (1946) (dissenting opinion). See 2 Wigmore, *Evidence*, § 463 (3d ed. 1940). Since comparison of similar property is necessary for valuation, survey methods could be used in accumulating and presenting in aggregate form data of comparable sales.

(*172*) McCoid, reference *160*, p. 235.

(*173*) *United States* v. *88 Cases*, 187 F. 2d 967 (3d Cir.), cert. denied, 342 U.S. 861 (1951); Dean, reference *167*, p. 5.

(*174*) Sorensen & Sorensen, reference *165*, p. 137.

(*175*) *Quaker Oats Co.* v. *General Mills, Inc.*, 134 F. 2d 429 (7th Cir. 1943); *Oneida, Ltd.* v. *National Silver Co.*, 25 N.Y.S. 2d 271 (Sup. Ct. 1940); *cf. Alexander Young Distilling Co.* v. *National Distillers Prod. Corp.*, 40 F. Supp. 748 (E.D. Pa. 1941).

(*176*) *Pretrial Practice in State Condemnation Cases for Highway Purposes*, by M. H. Naftalin, in *Highway Laws, 1961*, Highway Research Board Bulletin 294, pp. 15-30; for a bibliography of articles on pretrial procedure, see Report of Comm. on Condemnation and Condemnation Procedure, Municipal Law Section, A.B.A., 1960, at 153. In a condemnation proceeding, a number of economic facts may be stipulated; for instance, the severance damage case studies or the economic impact study findings could be stipulated as factual materials to which there would be no objection. Thus, a struggle over the adequacy or inadequacy of the data may be avoided. In this fashion, solid, factual materials may be admitted on stipulation, thereby narrowing wide disparities in land estimates through the mutual agreement in use of research materials.

(*177*) Submitting such a report to opposing counsel does not include the work product of the propouent of the report. It is discoverable by the other side only if there are special circumstances that make it essential to the preparation of his case and in the interest of justice that the statements be produced for his inspection or copying. See *Hickman* v. *Taylor*, 329 U.S. 495 (1947); *Walsh* v. *Reynolds Metals Co.*, 15 F.R.D. 376 (D. N. J. 1954); see generally Luttrell, *Some Applicable Rules in the Trial of a Condemnation Case*, 28 Appraisal J. 213, 216 (1960).

(*178*) Kennedy, *Law and the Courts*, in *The Polls and Public Opinion*, pp. 92, 101 (1949); Comment, 30 Tex. L. Rev., pp. 112, 118 (1951).

(*179*) Kennedy reference *178*, p. 101; Sorensen & Sorensen, reference *165*, pp. 134 *et seq.*

(*180*) Barksdale, *Use of Survey Research Findings as Legal Evidence*, p. xiii (1957).

(*181*) *United States* v. *Magyar*, 273 F. 2d 412 (2d Cir. 1959); *State* v. *Hunter*, 270 Ala. 57, 116 So. 2d 383 (1959); *Arkansas State Highway Commission* v. *Addy*, 329 S.W. 2d 535 (Ark. 1959); *Arkansas State Highway Commission* v. *Huges*, 328 S.W. 2d 391 (Ark. 1959); *Skinner* v. *Polk County*, 250 Iowa 1264, 98 N.W. 2d 749 (1959); *Stortenbecker* v. *Iowa Power & Light Co.*, 250 Iowa 1073, 96 N.W. 2d 468 (1959); *Luecke* v. *State Highway Commission*, 186 Kans. 584, 352 P. 2d 454 (1960); *United Fuel Gas Co.* v. *Mauk*, 325 S.W. 2d 339 (Ky. 1959); *Mississippi State Highway Commission* v. *Peterson*, 117 So. 2d 452 (Miss. 1960); *Mississippi State Highway Commission* v. *Pittman*, 238 Miss. 402, 117 So. 2d 197 (1960); *Mississippi State Highway Commission* v. *Ellzey*, 237 Miss. 345, 114 So. 2d 769 (1959); *Mississippi State Highway Commission* v. *Taylor*, 237 Miss. 847, 116 So. 2d 757 (1959); *Clark County School Dist.* v. *Mueller*, 348 P. 2d 164 (Nev. 1960); *Allbro* v. *Vallone*, 158 P. 2d 571 (R.I. 1960); *State* v. *Coffield*, 328 S.W. 2d 916 (Tex. 1959); *Utech* v. *City of Milwaukee*, 9 Wis. 2d 352, 101 N.W. 2d 57 (1960).

(*182*) *Arkansas State Highway Commission* v. *Addy*, 329 S.W. 2d 535 (Ark. 1959); *United Fuel Gas Co.* v. *Mauk*, 325 S.W. 2d 339 (Ky. 1959); *Mississippi State Highway Commission* v. *Taylor*, 237 Miss. 847, 116 So. 2d 757 (Miss. 1959).

(*183*) *Clark County School Dist.* v. *Mueller*, 348 P. 2d 164 (Nev. 1960); *Allbro* v. *Vallone*, 158 A. 2d 571 (R.I. 1960); *Utech* v. *City of Milwaukee*, 9 Wis. 2d 352, 101 N.W. 2d 57 (1960).

APPENDIX I

States Whose Constitutions Require Compensation

Part A, For Taking Property by Eminent Domain

Alabama:[1] Ala. Const., art. 1, § 23.
Connecticut: Conn. Const., art. 1, § 11.
Delaware: Dela. Const., art. I, § 8.
Florida: Fla. Const., Declar. of Rts., § 12.
Hawaii: Hawaii Const., art 1, § 18.
Idaho: Idaho Const., art. 1, § 14.
Indiana: Ind. Const., art. 1, § 21.
Iowa: Iowa Const., art. 1, § 18.
Kansas:[2] Kans. Const., art. 12, § 4 (not applicable to the State or public corporations).
Kentucky:[1] Ky. Const., § 13.
Maine: Maine Const., art 1, § 21.
Massachusetts: Mass. Const., pt. 1, art. 10.
Maryland: Md. Const., art. 3, § 40.
Michigan: Mich. Const., art. 13, § 1.
Nevada: Nev. Const., art. 1, § 8.
New Hampshire:[3] N.H. Const., pt. 1, art. 12 (by implication as construed, *Great Falls Mfg. Co.* v. *Vernald*, 47 N.H. 444, 455 (1867).
New Jersey: N.J. Const., pt. 20, art. 1.
New York: N.Y. Const., art. 1, § 7.
Ohio: Ohio Const., art. 1, § 19.
Oregon: Oreg. Const., art. 1, § 18.
Pennsylvania:[1] Pa. Const., art. 1, § 10.
Rhode Island: R.I. Const., art. 1, § 16.
South Carolina: S.C. Const., art. 1, § 17.
Tennessee: Tenn. Const., art. 1, § 21.
Vermont: Vt. Const., ch. 1, art. 2.
Wisconsin: Wis. Const., art. 1, § 13.

Part B, For Taking or Damaging Property by Eminent Domain

Alabama:[4] Ala. Const., art. 12, § 235 (where a municipal or other corporation is condemning).
Alaska: Alaska Const., art. 1, § 18.
Arizona: Ariz. Const., art. 2, § 17.
Arkansas: Ark. Const., art. 2, § 22.
California: Calif. Const., art. 1, § 14.
Colorado: Colo. Const., art. 2, § 15.
Georgia: Ga. Const., art 1, § 3, par. 1.
Illinois: Ill. Const., art. 2, § 13.
Kentucky:[4] Ky. Const., § 242 (where a municipal or other corporation is condemning).
Louisiana: La. Coust., art. 1, § 2.
Minnesota: Minn. Const., art. 1, § 13.
Mississippi: Miss. Const., art. 3, § 17.
Missouri: Mo. Const., art. 1, § 25.
Montana: Mont. Const., art. 3, § 14.
Nebraska: Nebr. Const., art. 1, § 21.
New Mexico: N. Mex. Const., art. 2, § 20.
North Dakota: N. Dak. Const., art. 1, § 14.
Oklahoma: Okla. Const., art. 2, § 24.
Pennsylvania:[4] Pa. Const., art. 16, § 8 (where a municipal or other corporation is condemning).
South Dakota: S. Dak. Coust., art. 6, § 13; S. Dak. Const., art. 17, § 18 (applicable to municipal and other corporations).
Texas: Tex. Const., art. 1, § 17.
Utah: Utah Const., art. 1, § 22.
Virginia: Va. Const., § 58.
Washington: Wash. Const., art. 1, § 8.
West Virginia: W. Va. Const., art. 3, § 9.
Wyoming: Wyo. Const., art. 1, § 33.

[1] Taking provisions applicable to all types of condemnation.

[2] No compensation provision applicable to the exercise of eminent domain by the State or a public corporation.

[3] Compensation requirement merely has been deemed to be implied by a consent provision.

[4] Taking or damaging provisions applicable to the exercise of eminent domain by the State or a public corporation.

APPENDIX II

Setoff Rules When a State or Local Government Takes Property for Highway Construction or Improvement

Part A, General and Special Benefits Against Value of Land Taken and Severance Damages

Alabama:[1] Ala. Const., art. 1, § 23, as construed in *McRea v. Marion County*, 222 Ala. 511, 133 So. 278 (1931); Ala. Code Ann., tit. 19, § 14 (1940), but see Part B.

New Mexico: *Board of Commissioners* v. *Gardner*, 57 N.M. 78, 260 P. 2d 682 (1953).

North Carolina: N.C. Gen. Stat., § 136-19 (1958), as construed in *Barnes* v. *North Carolina State Highway Commission*, 250 N.C. 378, 109 S.E. 219 (1959).

South Carolina:[1] S.C. Code, by §§ 25-165 (Supp. 1960), 33-127, 33-136 (1952), as amended; see generally *Smith* v. *City of Greenville*, 229 S.C. 252, 92 S.E. 2d 639 (1956).

Part B, Special Benefits Only Against Value of Land Taken and Severance Damages

Alabama[2] (highway improvements by local governments): Ala. Const., art. 12, § 223, as distinguished in *McRea* v. *Marion County*, 222 Ala. 511, 133 So. 278 (1931).

Arkansas Ark. Stat. Ann., § 76-521 (1947); *Ball* v. *Independence County*, 214 Ark. 694, 217 S.W. 2d 913 (1949).

Connecticut: Conn. Gen. Stat., 13-145 (1958); *Sorenson* v. *Fox*, 132 Conn. 583, 46 A. 2d 125 (1946); *Schwartz* v. *City of New London*, 20 Conn. Supp. 21, 120A. 2d 84 (1955).

Delaware:[3] *State ex rel. State Highway Department* v. *Morris*, 47 Del. 477, 93 A. 2d 523 (Super. Ct. 1952).

Florida: Fla. Stat., § 73.10(3) (1957).

Hawaii:[4] Hawaii Rev. Laws, § 8-21 (1955) (except in road widening or realinement cases); but see Part D.

Kansas: Kans. Gen. Stat., §§ 26-209, 68-706 (1949), as amended; *Trasper* v. *Board of Commissioners*, 27 Kans. 391 (1882).

Maine: *Boober* v. *Towne*, 127 Maine 332, 143 Atl. 176 (1928); *In re Penley*, 89 Maine 313, 36 Atl. 397 (1896).

Massachusetts: Mass. Gen. Laws Ann., ch. 79, § 12 (1958).

Michigan:[3] Mich. Stat. Ann., c. 64, § 8.189 (1958).

New Hampshire: *Whitcher* v. *Benton*, 50 N.H. 25 (1870).

New Jersey: *State* v. *Hudson County Board of Chosen Freeholders*, 55 N.J.L. 88, 25 Atl. 322 (1892).

Minnesota: *Chicago, R.I. and P. Ry.* v. *City of Minneapolis*, 164 Minn. 226, 205 N.W. 640 (1925).

Pennsylvania:[5] Johnson's Petition, 344 Pa. 5, 23 A. 2d 880 (1942).

Rhode Island: *D'Angelo* v. *Director of Public Works*, 152 A. 2d 211 (R.I. 1959).

South Carolina[2] (condemnation by county government): S.C. Code, § 33-840 (1952), as distinguished in *Smith* v. *City of Greenville*, 229 S.C. 252, 92 S.E. 2d 639 (1956).

South Dakota:[3] S.D. Code, §§ 28.13 A09, 37.4010 (Supp. 1960).

Vermont:[5] Vt. Stat. Ann., tit. 19, § 221 (1959).

Washington: Wash. Rev. Code, §§ 8.04.080, 8.08.040, 8.12.190 (1961).

Part C, General and Special Benefits Against Severance Damages Only

New York: *Hartman* v. *State*, 5 Misc. 2d 636, 161 N.Y.S. 2d 748 (Ct. Cl. 1957); New York, *W. & B. Ry.* v. *Siebrecht*, 73 Misc. 219, 130 N.Y.S. 1005 (Sup. Ct. 1919).

Virginia: Va. Code Ann., § 33-73 (1950), as construed in *Long* v. *Shirley*, 117 Va. 401, 14 S.E. 2d 375 (1951).

West Virginia: W. Va. Code, § 5380 (1955) as construed in *Strouds Creek & M.R.R.* v. *Herald*, 131 W. Va. 45, 45 S.E. 2d 513 (1947).

Part D, Special Benefits Against Severance Damages Only

Alaska:[6] Alaska Comp. Laws Ann., § 57-7-13 (1949).

Arizona:[6] Ariz. Rev. Stat. Ann., § 12-1122 (1956), as construed in *Pima County* v. *De Concini*, 79 Ariz. 154, 285 P. 2d 609 (1955).

California: Calif. Civil Procedure Code, § 1248, as construed in *People* v. *Schultz Co.*, 123 Calif. App. 2d 925, 268 P. 2d 117 (1954).

Colorado: Colo. Rev. Stat., § 50-1-17 (1953); *Denver Joint Stock Land Bank* v. *Board of County Commissioners*, 105 Colo. 366, 98 P. 2d 283 (1940).

Georgia: Ga. Code Ann., § 36-504 (1933), as construed in *State Highway Board* v. *Pridges*, 60 Ga. App. 240, 3 S.E. 2d 907 (1939).

Idaho: Idaho Code, § 7-711 (1947).

Hawaii:[7] Hawaii Rev. Laws, § 8-21 (1955) (in road widening or realinement cases only).

Illinois: Ill. Const., art. 2, § 13, as construed in *Kane* v. *City of Chicago*, 392 Ill. 172, 64 N.E. 2d 506 (1945); *Department of Public Works and Buildings* v. *Barton*, 371 Ill. 11, 19 N.E. 2d 935 (1939).

Indiana: Burns Ind. Stat. Ann., § 3-1706 (1946), as construed in *State* v. *Smith*, 237 Ind. 72, 143 N.E. 2d 666 (1957).

Kentucky: Ky. Rev. Stat., §§177.083, 416.100-416.120, 416.230-416.240 (1960); *Freuel* v. *Commonwealth*, 331 S.W. 2d 710 (1959).

Louisiana: *Louisiana Highway Commission* v. *Grey*, 197 La. 942, 2 So. 2d 654 (1941).

Maryland: Md. Ann. Code, art. 33A, § 25 (1957); *Pumphrey* v. *State Roads Commission*, 175 Md. 498, 2 A. 2d 668 (1937).

Mississippi: *Mississippi State Highway Commission* v. *Hillman*, 189 Miss. 859, 198 So. 565 (1940).

Missouri: Mo. Rev. Stat., § 227.120 (1959).

Montana: Mont. Rev. Code, § 99-9912 (1949), as amended.

Nebraska: *Crawford* v. *Central Neb. Public Power & Irr. Dist.*, 154 Nebr. 832, 49 N.W. 2d 682 (1951).

Nevada:[6] Nev. Rev. Stat., § 37.110 (1960).

North Dakota: N. Dak. Cent. Code, § 35-15-22 (1960), as construed in *Lineburg* v. *Sandoen*, 74 N. Dak. 364, 21 N.W. 2d 808 (1946).

Ohio: Ohio Const., art 1, § 19; *In re Abraham*, 121 N.E. 2d 695 (Ohio Com. Pl. 1953).

Oregon: *State Highway Commission* v. *Bailey*, 212 Oreg. 261, 319 P. 2d 906 (1957).

Tennessee: Tenn. Code Ann. § 23-1414 (1955).

Texas: Tex. Civ. Stat., art. 3265 (1952), as construed in *State* v. *Carpenter*, 126 Tex. 604, 89 S.W. 2d 194 (1936).

Utah: Utah Code Ann., § 104-61-11 (1943).

Wisconsin: Wis. Stat. Ann., § 32.09 (Supp. 1961).

Wyoming:[3] Wyo. Stat., § 1-775 (1957).

Part E, Setoff Prohibited

Iowa: Iowa Const., art. 1, § 18.

Oklahoma: Okla. Const., art. 2, § 24.

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[1] In certain cases, only special benefits may be setoff. See Part B.

[2] See Part A for general rule.

[3] Setoff statutes seem to contain sufficiently broad language to authorize general benefit setoff, if and when they should be construed on this point.

[4] In certain cases, setoff is allowed only against severance damages, see Part D.

[5] Setoff against full value is implied from the use of the before-and-after formula in these jurisdictions.

[6] Although not yet so construed, these statutes are identical to the California provision, which is limited to special benefits.

[7] See Part B.

(*Appendix III appears on p. 40*)

Motor Vehicle Size and Weight Limits

A comparison of State legal limits of motor-vehicle sizes and weights with standards recommended by the American Association of State Highway Officials is given in the table on pages 36-37. The statutory limits reported in this tabulation, prepared by the Bureau of Public Roads as of December 31, 1961, have been reviewed for accuracy by the appropriate State officials.

Statutory limits are shown for width, height, and length of vehicles; number of towed units; maximum axle loads for single and tandem axles; and maximum gross weights for single-unit truck, truck-tractor semitrailer combinations, and other combinations.

APPENDIX III

Admissibility of Comparable Sales as Evidence of Market Value in Condemnation Proceedings

Part A. Independently Admissible as Evidence of Market Value

Alabama: *Southern Elec. Generating Company* v. *Leibacher*, 269 Ala. 9, 110 So. 2d 308 (1959).

Arizona: *Town of Williams* v. *Perrin*, 70 Ariz. 157, 217, P. 2d 918 (1950).

Arkansas: *Sewer & Water Works Improvement Dist. No. 1* v. *McClendon*, 187 Ark. 510, 60 S.W. 2d 920 (1933).

California: *County of Los Angeles* v. *Faus*, 48 Calif. 2d 672, 312 P. 2d 680 (1957).

Colorado: *Kistler* v. *Northern Colo. Water Conservancy Dist.*, 126 Colo. 11, 246 P. 2d 616 (1952).

Connecticut: *Campbell* v. *City of New Haven*, 101 Conn. 173, 125 Atl. 650 (1924).

Delaware: *Wilmington Housing Authority* v. *Harris*, 47 Del. 469, 93 A. 2d 518 (Super. Ct. 1952).

Florida: *City of Tampa* v. *Texas Company*, 107 So. 2d 216 (Fla. App. 1958).

Georgia: *Flemister* v. *Central Ga. Power Company*, 140 Ga. 511, 79 S.E. 148 (1913); *Fulton County* v. *Cox*, 109 S.E. 2d 849 (Ga. App. 1959).

Illinois: *City of Chicago* v. *Blanton*, 15 Ill. 2d 198, 154 N.E. 2d 242 (1958).

Indiana: *Northern Ind. Pub. Serv. Company* v. *Darling*, 239 Ind. 237, 154 N.E. 2d 881 (1958).

Iowa: *Redfield* v. *Iowa State Highway Commission*, 251 Iowa 332, 99 N.W. 2d 413 (1959).

Kansas: *Wood* v. *Syracuse School Dist.* 108 Kans. 1, 193 P. 1049 (1920).

Kentucky: *Stewart* v. *Commonwealth*, 337 S.W. 2d 880 (Ky. 1960).

Louisiana: *State* v. *Havard*, 239 La. 133, 118 So. 2d 131 (1960).

Maryland: *Patterson* v. *Mayor & City Council of Baltimore*, 127 Md. 233, 96 Atl. 458 (1915).

Massachusetts: *Epstein* v. *Boston Housing Authority*, 317 Mass. 297, 58 N.E. 2d 135 (1944).

Missouri: *State* v. *Bruening*, 326 S.W. 2d 305 (Mo. 1959).

Nebraska: *Langdon* v. *Loup River Public Power Dist.*, 142 Nebr. 859, 8 N.W. 2d 201 (1943).

New Hampshire: *Eames* v. *Southern N.H. Hydro-Elect. Corp.*, 85 N.H. 379, 159 Atl. 128 (1932).

New Jersey: *Curley* v. *Mayor & Aldermen of Jersey City*, 83 N.J.L. 760, 85 Atl. 197 (E. & A. 1912); *State* v. *Williams*, 65 N.J. Super. 518, 168 A. 2d 233 (App. Div. 1961).

New York: *Village of Lawrence* v. *Greenwood*, 300 N.Y. 231, 90 N.E. 2d 53 (1949).

Oregon: *State* v. *Parker*, Oreg., 357 P. 2d 548, (1960).

Tennessee: *Union Ry.* v. *Hunton*, 114 Tenn. 609, 88 S.W. 182 (1905).

Texas: *City of Austin* v. *Canizzo*, 153 Tex. 324, 267 S.W. 2d 808 (1954).

Utah: *State* v. *Peek*, 1 Utah 2d 263, 265 P. 2d 630 (1953).

Virginia: *May* v. *Dewey*, 201 Va. 621, 112 S.E. 2d 838 (1960).

Washington: *Seattle & M. Ry.* v. *Gilchrist*, 4 Wash. 509, 30 Pac. 738 (1892).

Wisconsin: *Blick* v. *Ozaukee County*, 180 Wis. 45, 192 N.W. 380 (1923).

Wyoming: *Morrison* v. *Cottonwood Dev. Co.*, 38 Wyo. 190, 266 P. 117 (1928).

Part B. Admissible in Support of Opinion Testimony

District of Columbia: *District of Columbia Redev. Land Agency* v. *61 Parcels of Land*, 98 U.S. App. D.C. 367, 235 F. 2d 864 (1956) (admissible to support appraiser's expert testimony but subject to the court's discretion).

Mississippi: *Mississippi State Highway Commission* v. *Rogers*, 236 Miss. 800, 122 So. 2d 250 (1959).

Ohio: *In re* Ohio Turnpike Commission, 164 Ohio St. 377, 131 N.E. 2d 397 (1955), cert. denied, 352 U.S. 806 (1957).

Part C, Judicial Indication That it Would be Independently Admissible Though Never so Held

Nevada: *Clark County School Dist.* v. *Mueller*, 76 Nev. 11, 348 P. 2d 164 (1960) (dictum for such evidence).

Oklahoma: *Durell* v. *Public Serv. Co.*, 174 Okla. 549, 51 2d 517 (1935) (rule stated as dictum).

Rhode Island: *Hervey* v. *City of Providence*, 47 R.I. 37 133 A. 618 (1926) (issue of remotemess held properly decid by judge to exclude evidence; Massachusetts rule assum to be determinative).

South Carolina: *Wateree Power Co.* v. *Rion*, 113 S.C. 30 102 S.E. 331 (1920) (seems to assure Mass. rule in holdi that sales to condemnor, where only sales of comparable la available, were admissible); *South Carolina Highway Depa ment* v. *Hines*, 234 S.C. 254, 107 S.E. 2d 643 (1959) (Ge Rule recognized without indication whether it was S.C. la (evidence excluded, because as a mere offer not accepte it was not within the rule).

West Virginia: (No cases dealing with evidence of comp able sales to noncondemnor); *cf. United Fuel Gas Co.* Allen, 137 W. Va. 897, 75 S.E. 2d 88 (1953) (sale to condemn voluntarily made is good where severance damages are n involved).

Part D. Admissible Only to Impeac Opinion Testimony

Michigan: *Lockeman* v. *Dillman*, 255 Mich. 152, 237 N.W 552 (1931).

Minnesota: *Minneapolis-St. Paul Sanitary Dist.* v. *Fi patrick*, 201 Minn. 442, 277 N.W. 394 (1937).

North Carolina: *Templeton* v. *State Highway Commissio* 118 S.E. 2d 918 (N.C. 1961).

Pennsylvania: *Serals* v. *West Chester Borough School D* 292 Pa. 134, 140 Atl. 632 (1928).

Part E, No Cases in Point

Alaska, Hawaii, Idaho, Maine, Montana, New Mexi North Dakota, South Dakota, Vermont.

APPENDIX IV

An Example of the Determination of General and Special Benefits

In figure 1:

Town *X*, with residential lots *a* and *b*, represents a hypothetical town affected by a new highway bypassing it.

Town *Y*, with residential lots *c* and *d*, represents a hypothetical town that is comparable to town *X* but not affected by new highway construction.

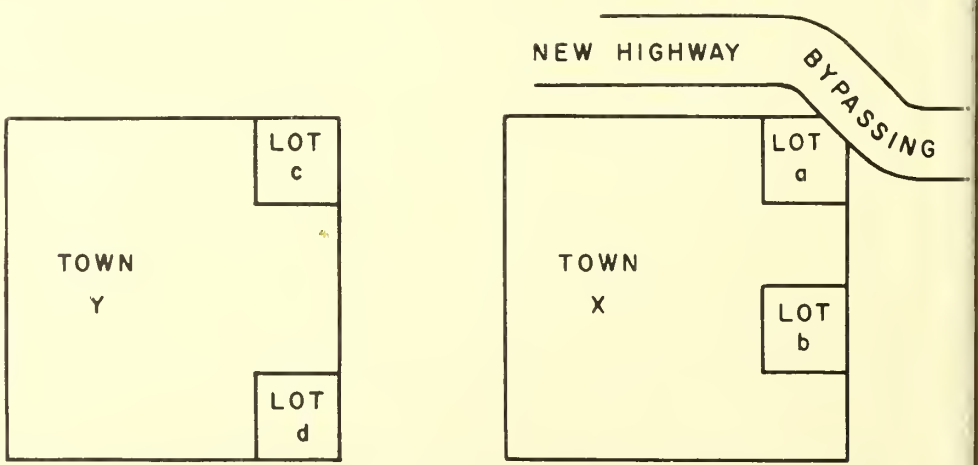

Figure 1.—Hypothetical towns and lots used to illustrate benefits from a highway improvement.

Benefits from Highway Improvement

Each of the four residential lots was valued at $1,000 prior to construction of the new highway bypass. After construction of the bypass, in town *X*, lot *a* had a value of $1,400 and lot *b* had a value of $1,200. In town *Y*, the value of lots *c* and *d* remained unchanged at $1,000 each. The change in value of lots in town *X* illustrates benefits of increased property values derived from construction of a new highway.

Property values in town *X* increased an average of $200 per lot following the opening of the bypass but no increase in property values occurred in the control town of *Y*. The average increase in value of $200 represents a general benefit for each of the lots *a* and *b*, $1,200 minus $1,000.

Within town *X*, lot *a*, which was partially taken for highway right-of-way, was affected by the highway construction to a greater extent than lot *b*, a comparable lot within the same community. After the opening of the highway, lot *a* sold for $1,400—thus, a special benefit of $200 accrued to lot *a*, the difference between $1,400 and $1,200.

For the purpose of determining benefits accruing to lot the control for special benefits is lot *b*; for general and spec benefits, the control is either lot *c* or *d*.

(*Appendix V appears on pp. 42–43*)

NEW PUBLICATIONS

Four recent publications by the Bureau of Public Roads are now available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at the prices indicated for each.

Aggregate Gradation for Highways

Aggregate Gradation for Highways (25¢), contains two important articles on this subject: *Aggregate Gradation: Simplification, Standardization, and Uniform Application*, by a special committee of experts in the Bureau of Public Roads, and *A New Graphical Chart for Evaluating Aggregate Gradation*, by J. F. Goode and T. A. Lufsey.

The first article points out the need for simplification, standardization, and uniform application of aggregate gradation specifications in the highway field, noting the benefits that would result from reduction from their present extreme diversity. It explains the value of the simplified practice recommendation system for aggregate gradation, which provides a reasonably limited number of standard specifications and uniformity in the number and sizes of sieves for use in specifying these gradations. The article discusses the existing AASHO and ASTM standard aggregate specifications and recommends their universal adoption, recognizing that nonconforming gradations may be necessary as special provisions or supplemental specifications.

The second article describes the development of a new aggregate gradation chart using for its horizontal scale a power function rather than the logarithm of the sieve openings. With this chart, maximum density curves plot as a straight line from zero percent passing zero theoretical sieve size to 100 percent at the maximum size, rather than the difficult-to-define, deeply sagging curves obtained by plotting on the customary gradation chart. The article demonstrates the value of the new chart in developing realistic specifications and in evaluating individual gradations, using as examples both actual field problems and laboratory experiments.

America's Lifelines—Federal Aid for Highways

America's Lifelines—Federal Aid for Highways (15¢), a colorful, illustrated leaflet, describes in simple terms the Federal-aid highway program and the functions of the Bureau of Public Roads. Information is included on the 41,000-mile National System of Interstate and Defense Highways (and a map of the system) and on the Federal-aid program for the improvement of the more extensive Federal-aid primary and secondary systems. Also described are the Bureau of Public Roads activities in roadbuilding on Federal lands, providing engineering services to other Federal agencies, highway planning, research, safety, and assistance to foreign governments in organizing highway departments and launching road improvement programs.

Increasing the Traffic-Carrying Capability of Urban Arterial Streets

Increasing the Traffic-Carrying Capability of Urban Arterial Streets (40¢), reported by Arthur A. Carter, Jr., describes a pilot study conducted by the Bureau of Public Roads of the traffic improvements that could be made on an urban arterial street, working within the existing right-of-way limits. The study was made in Washington, D.C., and has often been referred to as "The Wisconsin Avenue Study." The forepart of the publication reviews every known, practical means of improving traffic movements on an arterial street. Application was then made (in theory) of these means, singly and in combination, to provide a traffic stream having maximum capacity and minimum friction while at the same time providing conditions conducive to patronage of the adjacent land services. Three phases were entailed, the first involving little or no cost, the second requiring moderate cost and some construction, and the third calling for major expenditures. The calculated effects of the theoretical ultimate improvements would have permitted peak-hour traffic volume increases of 100 to 200 percent and increases in average speed from 14 20 to 25–30 miles per hour. The publication is not intended to be a manual, but should serve as an invaluable guide to highway and traffic engineers.

Manual for Highway Severance Damage Studies

The *Manual for Highway Severance Damage Studies* ($1.00) was prepared by the Highway and Land Administration Division, Bureau of Public Roads, in 1961, to serve as a guide particularly for use by the State highway departments. Widespread subsequent interest indicated the desirability of making the manual available through the Government Printing Office. Partial takings of property for highway right-of-way frequently involve severance damages which, without sufficient comparable information, are often difficult to evaluate. This manual calls attention to the value of information about actual experience in individual severance damage cases, and describes in some detail systematic methods of collecting, processing, and analyzing such information. A large body of facts on this subject will provide an invaluable reference "bank," both for economic research studies and for use in making and supporting sound appraisals of severance damages in actual right-of-way taking cases.

Errata

In the April 1962 issue of PUBLIC ROADS, vol. 32, No. 1, an error appears in the legend for figure 6, page 7, of the article, *Social Effects of Modern Highway Transportation*. The correct identification for the university in the center of the illustration is: University of Kansas City, Kansas City, Mo.

APPENDIX V

Status of Severance Damage Studies [1]

(As of September 1, 1961)

State	Research agency	Nature of study in progress	Studies completed
Alabama [2]	Alabama State Highway Department	Case studies of severance damages	
Arizona [2]	Arizona Highway Department	Analysis of cost data in connection with the acquisition of right-of-way for highway improvements.	
Arkansas [2]	Arkansas State Highway Commission		
California	California Division of Highways	Continuing case studies of severance damages to remainder properties after partial takings for right-of-way.	*Land Economic Studies, Remainder Parcel Analysis* No. 1—summarizes 10 remainder parcel sales in Vallejo, Calif. *Land Economic Studies*—summarizes 20 remainder parcel cases. *California Land Economic Studies-Techniques.* *Remainder Parcels*, a report of the Land Economics Study Section.
Colorado	Colorado Department of Highways	Severance damages, right-of-way acquisition, and partial takings, includiug case studies of same.	*Case Studies of Damage Payments*, Nos. 1 through 21.
Florida [2]	Florida State Road Department		
Georgia [2]	Georgia State Highway Department	Analysis of factual evidence with respect to values fixed, payments made, disposition of remainder properties, and use of remainder properties.	
Hawaii [2]	Hawaii Department of Transportation, Division of Highways.		
Idaho	Idaho Department of Highways	Analysis of actual damage as compared with damage awards in connection with right-of-way takings.	
Illinois	University of Illinois	Case studies of land values and severance damages to remainder properties after partial takings for right-of-way.	*Land Economic Studies*, Nos. 1–5.
Indiana	Indiana State Highway Department	Evaluation of right-of-way appraisal values and determination of a series of basic uniform rules and guides to be used in land appraisals.	
Iowa	Iowa State Highway Commission	Effects of farm unit severance resulting from right-of-way purchase	
Kansas	State Highway Commission of Kansas	Severance damage studies as part of a larger economic impact study.	
Kentucky [2]	Kentucky Department of Highways	Investigation and evaluation of damage effects in terms of market value of a highway building program on remainders of partial takings in urban and rural areas.	
Louisiana [2]	Louisiana Department of Highways		
Maine	Maine State Highway Commission	Case study of partial taking and severance damage	
Maryland	Maryland State Roads Commission	Case studies of remainder properties after purchase for right-of-way.	
Michigan	Michigan State Highway Department	Guide for right-of-way appraisers in estimating costs for property acquired for highway right-of-way.	*Land Economic Studies*, Nos. 1–7.
Minnesota	University of Minnesota	Relationships between compensation payments and the extent of the property taken plus damages as a direct consequence of the highway.	*How Farmers Adjusted to an Interstate Highway in Minnesota.*
	Minnesota Department of Highways	Analysis of severance damages as part of an economic impact study being made on a segment of I-094-3(15) in St. Paul.	
Mississippi	University of Mississippi	Analysis of effects on land use, land value, and fragmentation Case studies of partial takings of rural properties	
Missouri	Missouri State Highway Commission	Severance damage studies being conducted as part of a larger economic impact study.	
Montana [2]	Montana State Highway Commission		

See footnotes at end of table.

APPENDIX V—Continued

Status of Severance Damage Studies[1]

State	Research agency	Nature of study in progress	Studies completed
Nebraska [2]	Nebraska Department of Roads		
New Jersey	New Jersey State Highway Department.	Develop data in order to provide a more reliable basis for estimating severance and consequential damage.	*Severance Study Manual.*
New Mexico	New Mexico State Highway Commission.	Severance damage studies	
New York	New York Department of Public Works.	Severance damage studies being conducted as part of the Northway economic impact study.	
North Carolina [2]			
North Dakota [2]	North Dakota State Highway Department.	Case studies of several sections of highway	
Ohio	Ohio Department of Highways	Studies of land values and relationship of subsequent sales prices of remainder parcels to "before" value, by type of remainder parcels.	
Oklahoma	Oklahoma State Highway Department.	Collection and interpretation of sales data on severed parcels of land previously acquired. These data are expected to provide a basis for right-of-way appraisers to substantiate "after" values in the "before and after" appraisals for highway right-of-way.	
Oregon	Oregon State Highway Commission	Case studies of land values and severance damages to remainder properties after partial takings for right-of-way.	*Land Economic Studies Properties Abutting Baldock Freeway.* *Oregon Land Economic Studies*, Nos. 30–35.
South Carolina [2]	University of South Carolina	Severance damage studies being conducted as part of a larger economic impact study.	
South Dakota	South Dakota Department of Highways.	Parcel by parcel analysis of remainder properties adjacent to completed segments of the Interstate System to determine effects of the facility on (1) the market value of remaining land, and (2) the development of the remaining land.	
Tennessee	University of Tennessee	Severance damage studies being conducted as part of a larger economic impact study.	
Utah [2]	Utah State Road Commission		
Texas	Texas Transportation Institute, Texas A and M College.	Various aspects connected with the acquisition of right-of-way for highway use, including studies of case histories of remainder parcels and effects of displacement of persons and investments resulting from right-of-way acquisitions.	
Vermont	Vermont Department of Highways	Provide a more reliable basis for estimating severance and consequential damage.	
Virginia	Virginia Department of Highways		A number of case histories have been completed.
Washington	Washington Department of Highways.	Analyze value of remainder properties after purchase of portion for highway use; case studies.	Individual land economic studies Nos. 1–28, 34, 36, and 41.
Wisconsin [2]	Wisconsin State Highway Commission.		
Wyoming [2]	Wyoming State Highway Commission.		
Nationwide	Agricultural Research Service, U.S. Department of Agriculture.		*The Effects on Farm Operating Units of Partial Taking for Controlled-Access Highways.*

[1] In most cases, these severance damage studies are being conducted by researchers within the State highway departments, although in a few instances the work is being done under contract. For additional information concerning any study, it is suggested that inquiry be made to the appropriate State highway department.

[2] In planning stage; others listed are underway.

Special Assessments in Theory and Practice

By FLOYD I. THIEL, Economist,
Economic Impact Research Branch,
Highway and Land Administration Division,
Bureau of Public Roads, and
H. RUSSELL BRIGGS,
Director, Economic Research Agency [1]

This article summarizes the report submitted on a study of the use of special assessments for financing public improvements and the relation of this type of financing to highway construction. The Bureau of Public Roads had this study made so that information could be presented to Congress on the special assessment procedure as an alternative method of financing highway construction. This study was conducted on a selective basis, and the utilization of the special assessment procedure was investigated in a restricted number of States by field investigations and through correspondence.

This summarization includes information on: The historical use of special assessment financing, the legal justification for use of this method in the United States, the prevalence of the use of the special assessment procedure, some of the purposes for which this type of financing is used, and some of the problems related to an equitable levying of special assessments, especially for highway construction.

Introduction

SPECIAL assessments are based on the principle of equity. The philosophy of the use of special assessments is that, the owner of property benefited by a public improvement should contribute toward the cost of the improvement, and taxpayers in general should not have to bear the expense of improvements of no particular advantage to them. The use of special assessments for financing certain public improvements such as highways or sewers is an indication that some local benefit or value can be attributed to the improvement— ". . . that the owner of a property assessed is to receive a benefit corresponding with the amount assessed." [2] The extent of this local or special benefit may be confined to abutting land and buildings or it may extend beyond, to noncontiguous property. In some instances, special assessments are used to finance public improvements affecting fairly large areas, for example, an entire city or several counties.

Special assessments, which may be imposed by central government bodies and/or special assessment districts, differ from general property taxes in several ways. Special assessments ordinarily apply to a more limited area for a specified duration of time. The typical special assessment applies to a fairly small area for a period of time that is only as long as is required to pay for the specific improvement; however, special assessments are used to some extent to cover the costs for certain repetitive undertakings. Another difference peculiar to special assessments is that property owners often initiate the application of such taxes by requesting special improvements. The special assessment procedure also is distinguished from the general property tax by the requirements for public hearings and notification thereof to all property owners who would be affected by the special assessment.

Special assessments formerly were in common use for financing rural roads—they still are used to a considerable extent for streets—and this financing procedure made a considerable contribution toward the early building of State highways. At the present time, interest in the use of special assessments as a method of public financing is related to the indirect connection this procedure has to financing the Nation's network of highways. The chief significance in the use of the special assessment procedure lies in the fact that it is a well-established and judicially sustained procedure for financing public investments by levies on property, with such levies designed to be in proportion to the benefits to be received from the improvement thus financed. A clear precedent, predating the practice of road-user taxation, has been established for taxation of the so-called nonuser in proportion to the benefits he will receive. Because of its past use for financing highways, the special assessment procedure was among those for which the Bureau of Public Roads gathered information in carrying out its responsibility to provide the Congress with information about alternative methods of highway financing in the highway cost allocation study.

Historical Background

Special assessments to pay the cost of public improvements that provided benefits for certain property are known to have been used as early as the 13th century in England, and the procedure may have been used even earlier. By 1800, use of the special assessment had become firmly entrenched as part of the American system of public financing; however, there has not been complete agreement as to the constitutional bases for use of this procedure.

When the right to levy special assessments first was tested judicially, some courts ruled that, as the procedure had become so inherent a part of the American system of public finance, the imposts should be sustained in the public interest. Other courts justified the use of special assessments on the basis that the authority to levy them was a right of eminent domain, a position that soon was abandoned.

Currently, special assessments for publ improvements that enhance the value of re property are being levied under the taxin power. Assessments for improvements deeme necessary for the public's health and safety however, are justified as a manifestation the police power; a principle accepted aft abandonment of the position on the rig of eminent domain. Special assessments f financing repetitive public undertakings, suc as for sidewalks and sewers, also may b sustained under the taxing power but, b cause in theory assessments for such purpos are levied primarily to abate a nuisance, the are justified more properly as manifestatio of the police power.

Advantages and Disadvantages

As a means for financing public improv ments, the special assessment levy has bo advantages and disadvantages—some aspec of the procedure might be regarded as eith or both. As special assessments general are not subject to tax or bonded indebtedne limitations, a political entity may undertak a program for needed public improvement in situations where limitations on spendin would not permit such improvements to b accomplished with general funds. Obviously a procedure that permits financing of neede public improvements on a timely basis ha certain advantages. At the same time, th possibility of overdevelopment through us of special assessment financing might prov to be disadvantageous—overdevelopment tha might occur because of the absence of som of the limitations imposed on general fun financing. Other possible advantages an disadvantages attendant on the use of speci assessment financing are discussed in th following paragraphs.

[1] This paper is a summarization of some of the more significant aspects of *Special Assessments in Theory and Practice*, a report prepared for the Bureau of Public Roads by the Economic Research Agency, Madison, Wis.

[2] *In re* Mead, 74 N.Y. 216.

Another advantage of the special assessment procedure is the feasibility of using it to levy charges for expensive facilities only against those affected, rather than against all taxpayers. Special assessments for such purposes generally originate at the request of, and/or have the approval of, those who will be called upon to meet the payments—the very essence of democracy. Because of rough terrain, poor drainage, low population densities, and other factors, the cost for public improvements such as streets or sewers may be particularly high in certain areas. Use of a special assessment to finance this type of work is considered equitable and advantageous. Special assessments also can be levied against religious, educational, charitable, and other types of property that are exempt from the general property tax. This feature of the special assessment procedure is regarded as an advantage insofar as providing money for public purposes is concerned.

Special assessment financing also is considered advantageous because its use permits completion of needed public improvements without causing year-to-year fluctuations in the general revenue taxation program. Thus, public improvements for a limited area can be financed without any impact on the general property taxes of an entire governmental unit.

A further advantage attributed to special assessment financing is the compatibility of this procedure for use with other methods of financing, including the general property tax.

Perhaps the major disadvantage of special assessment financing is that facilities may be extended beyond reasonable and legitimate needs; this disadvantage was referred to previously in connection with fund limitations. Overextension of facilities poses a danger that payments may become delinquent, a danger that experience with special assessments has shown to be very real. During the rapid-growth period of the 1920's, special assessment financing was used to a considerable extent to meet the demand for improved roads and streets. Overexpansion and overdevelopment occurred in some areas and, in the late 1920's and during the 1930's, widespread delinquency in payments of special assessments existed. For example, out of $17 million of special assessment bonds in Cleveland, Ohio, in 1929 delinquent payments totaled more than $10 million. In California in 1936, of about $63 million in special assessment bonds outstanding in Los Angeles County, payments were delinquent on more than a third; in San Diego County, nearly $9 million out of about $14 million of special assessment bonds were in a default status.

The difficulty related to equitably apportioning benefits to particular properties is considered a problem or disadvantage in the use of the special assessment procedure. While considerable ingenuity has been demonstrated in assigning benefit and special assessment amounts to particular properties, these levies are imposed mainly by rule of thumb and guesswork. The fairly common procedure of charging abutting owners more than owners whose property is somewhat removed from the facility has certain shortcomings, which are obvious. In the case of a highway, for example, a property a block or more away may benefit as much as the property abutting the facility. In fact, property removed from the highway may sometimes benefit more than abutting property.

Additional disadvantages in the use of the special assessment procedure include the problems of administration; these problems have been suggested in the discussion on delinquent payments of special assessments. Special assessment levies commonly are made by local boards, which may lack the necessary administrative abilities. Because of this lack, and for other reasons—for example, borrowing for projects financed through special assessment procedures normally is quite expensive unless full faith and credit bonds are issued—special assessment financing may lead to inflated costs. Furthermore, presumably the costs of improvements enhancing property values eventually can be recovered automatically through the increased revenues obtained from general property taxes; consideration of this possibility may weigh against the use of special assessment procedures for financing certain improvements.

Use of Special Assessments

Some of the variations in the use of special assessment procedures have been referred to in foregoing paragraphs. The extent to which special assessments have been used has changed from year to year. Variations also have occurred in the use that different localities or areas make of special assessment financing, and considerable variety has been noted in the purposes for which special assessments have been and are being made.

The apogee of special assessment financing in the United States appears to have been reached in the 1920's. A marked decline in the use of special assessments began in the 1930's and continued until after World War II to about 1950. A general increase in the use of this method of financing public improvements began again with the 1950's, and the amount of revenue being raised by special assessments now equals that of the 1920's; however, this amount does not represent as high a percentage of total municipal revenues as was the case in the 1920's. During the middle 1950's, the relation of special assessments to general property taxes became quite stable, according to information compiled by the U.S. Census Bureau. With assessments spread on an *ad valorem* basis and those levied for continuing activities such as street maintenance and lighting excluded, revenue raised from special assessments during the period from 1954 through 1958 has averaged 2.5 percent of the revenues obtained from local property taxes.

Circumstances surrounding the use of special assessments vary from locality to locality. Wide variations have been found (1) in the method in which special assessments are levied, (2) in the size of the special assessment districts, and (3) in the extent to which various localities depend upon special assessments for the financing of public improvements.

Levying Special Assessments

A charge on a front foot basis appears to be the most common method employed in levying special assessments for public improvements such as streets or parks, with the charge often being higher for property near the facility. Some of the problems arising from the assumption that the amount of benefit accruing to a property varies directly with its proximity to the improvement or new facility were referred to previously. Attempts made to evolve an equitable basis for levying special assessments no doubt have been responsible for many of the additions introduced into special assessment procedures. Special assessments now are levied, quite commonly, on a combination of two or more factors, such as: (1) proximity to the facility, (2) frontage abutting the improvement, (3) frontage abutting a street within a specified area, (4) area of the property involved, (5) value of the property, (6) a fixed rate per lot or per other land parcel. In some instances, where major arterials are provided for only partially by special assessments, abutting property is charged only with the cost of providing normal paving, which is considered to be a 12-foot lane. In some cities such as Milwaukee, Wis., and Detroit, Mich., the special assessment levy is based on benefits that are assumed to apply to property that extends halfway to the next parallel street. In one mountainous section of Los Angeles where lots were irregular in shape, it was deemed more equitable to levy the special assessments in part on the value of the lots rather than on a frontage basis.

Sizes of Special Assessment Districts

The sizes of special assessment districts vary a great deal; the size generally is dependent on the extent of the supposed benefit and, therefore, on the nature of the improvement. Special assessment districts traditionally have been considered to abut the improvement being financed but, as previously noted, tradition often is not followed. In Omaha, Nebr., special assessments for a boulevard were levied against property three-fourths of a mile away. In Fremont, Nebr., the cost of a bridge was recovered by a special assessment levied against the entire town. Financing the Moffat Tunnel under the Continental Divide in Colorado involved the establishment of a special assessment district some 145 miles long that varied in width from 6 miles up to the width of an entire county, 40 miles or more.

Dependence on Special Assessments for Public Improvements

Differences in the extent to which various localities depend upon special assessments for the financing of public improvements can be seen by comparing the magnitude of special assessments for different areas or by relating levies for special assessments to general property taxes. In California, the San Diego $5 annual per capita charge for special assessments suggests that this method

of financing public improvements is depended upon to a greater extent than it is in Los Angeles, where the annual per capita charge for special assessments averages $2.55. In San Francisco, the dependence upon special assessment financing is even less, the annual per capita charge for special assessments being about 19 cents. A comparable disparity in the degree to which cities rely on special assessment financing has been noted in Indiana: the annual per capita charge varies from about $3 in Fort Wayne to only 6 cents in Terre Haute.

Comparison of the amounts raised by special assessments and by general property taxes provides another indication of the important reliance that some communities place on the use of special assessments. In some cases, special assessment levies have equaled or exceeded revenues from general property taxes although, as has been indicated, the ratio between revenues raised by property taxes and by special assessments has been quite constant for the country as a whole during recent years. For example, in a number of communities such as North Hempstead and Tonawanda, N. Y., and Gilroy and Los Attos, Calif., revenues in 1957–58 from special assessment levies were almost double those obtained from general property taxes.

Use of special assessments is fairly common; this method of financing is being used or is available for use in most American cities. In a survey of cities with populations of more than 10,000, conducted in 1959 by the International City Managers' Association, nearly 80 percent of the cities responding reported some use of special assessments. More than a third of the 14,472 autonomous special districts, excluding school districts, have the authority to levy special assessments. In California alone, some 2,982 special districts have authority to levy special assessments. In New York City, in 1958, the Board of Assessors approved 100 sewer and 97 highway projects, thereby causing the creation of 197 special assessment districts.

Table 1.—Receipts for highways from property imposts

Year	General property tax (1,000 dollars)	Special assessments	
		Amount, 1,000 dollars	Percentage of general tax
URBAN			
1957	238,270	104,807	44
1956	159,120	98,756	62
1955	152,653	86,887	57
1954	144,796	76,719	53
1953	123,949	71,870	58
1952	123,013	50,847	41
1951	131,112	41,701	32
1950	113,610	39,879	35
1949	90,835	41,800	46
RURAL			
1957	472,327	11,387	2
1956	454,030	9,911	2
1955	428,198	9,908	2
1954	413,859	6,360	2
1953	397,544	5,821	1
1952	370,648	5,942	2
1951	337,824	[1] 4,980	1
1950	329,475	5,080	2
1949	277,986	[2] 2,891	1

[1] Of this amount, $2,262,000 was spent in Illinois.
[2] Of this amount, $2,390,000 was spent in Illinois.

Purpose of Special Assessment

The International City Managers' Association survey, previously mentioned, also provided information on the purposes for which special assessments are used. More than 700 of the 876 cities responding indicated that special assessment levies were used to finance the paving of new streets. Some of the other major purposes for which special assessments were used included financing for curbs and gutters, sanitary sewers, and sidewalks.

The major purposes for which special assessments were used and the number of cities employing this financing procedure for each purpose are shown in the following list.

Purpose	*No. of cities*
New street paving	712
Curbs and gutters	665
Sanitary sewers	641
Sidewalks	631
Storm sewers	287
Waterlines	261
Repaving streets and alleys	222
Street lighting facilities	152
Off-street parking	68
Miscellaneous improvements	34

Special assessments are utilized for many purposes other than the fairly common ones indicated in the foregoing list. In some instances, special assessments are used for financing both the original cost of an improvement and its maintenance costs. Often they are financed partially by special assessment levies against property near the facility and partially by general property taxes.

The wide variety of purposes for which special assessments are levied is indicated by the 66 different functions or facilities that have been financed by special assessments in California. These ranged from the more common facilities and activities such as libraries, highway lighting, and fire protection—with more than 400 special assessment districts for each of these—to activities and facilities such as debris basin maintenance, mosquito abatement, parking, citrous pest control, separation of grade, transit, and water storage. In Los Angeles, an ordinance requires the demolition of dangerous and obsolete buildings and, in the absence of compliance therewith, permits this work to be done by the city and its cost recovered by a special assessment against the land. Special assessment procedures have even received some consideration as a means of financing slum clearance programs. Justification for such a use would be similar to that for other special assessment proposals—that the surrounding area, because of the benefits from the slum clearance, could be expected to meet at least part of the cost of the program.

As might be expected, purposes for which special assessments are used have changed over the years. Thirty years ago special assessments were used to a considerable extent for financing rural roads; today they are being used increasingly to finance street lighting, particularly ornamental lights, and for rehabilitation of downtown areas. Special assessments for rehabilitation projects may be imposed for such specific purposes as parking lots, malls, parks, etc. For example, to rehabilitate a downtown area, Royal Oak, Mich., used special assessment financing to provide a series of parking lots and malls. In this case, half the cost was met by an *ad valorem* levy against the downtown district and half by a graduated front foot charge.

Special Assessments for Highways

Highways and streets are quite analogous to such public activities as waterworks and sewers. While these facilities ordinarily provide more benefit to those using them than to other people in the community, most people, whether users or nonusers, are benefited to some degree from the existence of the facility. It has been deemed reasonable, therefore, to charge both users and nonusers for construction of the facility. A fairly common procedure, used for allocating the cost of major streets or highways between highway users and abutting owners, is to charge abutting property owners for a portion of the cost of arterial facilities equal to the cost of an access street or road. In the case of major highway improvements, the benefit deemed to accrue may be fairly widespread. For example, when Chicago's 12th Street was widened at a cost of $3.3 million, properties within an area of a little more than 5 square miles were deemed to be benefited and special assessments that amounted to nearly half the original cost were levied against the properties—special assessments totaled $1 million.

The nonuser charge may be imposed either as a property tax or a special assessment, or both. The fairly stable relationship between the total amount of municipal funds being raised as general revenues and as special assessments has been referred to previously. As shown in table 1, the relationship between the amounts that municipalities raise for highways as general revenues and as special assessments also has been fairly stable in recent years.

Whether special assessments can be used as a means of providing equitable financing for a public activity depends primarily on the extent of the benefits involved. The American public and the courts have accepted the principle that, as a local street benefits adjacent property, it is equitable and reasonable to expect the property owners involved to pay at least a portion of the cost. As it generally has been recognized that the benefits provided by a major highway extend beyond the limits of the abutting property. Now, more than in the past, highways are built primarily to serve highway users. Thus, while boulevards, parkways, and other main arterials formerly were financed partially by special assessments—and to some extent they are; for example, in Minneapolis and St. Paul, Minn., Milwaukee, Wis., and Detroit, Mich.—major highways now are being financed primarily from user revenue.

Application of Infrared Spectroscopy to Bituminous Mineral Filler Evaluation

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by BERNARD CHAIKEN, Chemist; WOODROW J. HALSTEAD, Supervisory Chemist; and ROBERT E. OLSEN, Highway Research Engineer

Problems encountered in classifying the minerals from material in a specific natural mineral deposit, which was intended for use as a filler in bituminous concrete, provided the basis for the research reported in this article. After chemical and petrographic examination of material from this deposit resulted in an uncertain classification, particularly as to whether kaolinite or other clay minerals were present, various techniques were employed by the Bureau of Public Roads in an attempt to classify the minerals and, more specifically, to determine the suitability of the material for bituminous construction. It is believed that the experiences from this research study will be helpful to others encountering a similar problem.

The methods used and the results obtained in this study are reported here. By resorting to a combination of instrumental methods, physical tests, and chemical analysis, the clay mineral nature of the material and its potential behavior as a filler for bituminous concrete were determined. The methods utilized in the study included infrared spectroscopy; x-ray diffraction; differential thermal analysis; fineness, surface area, and plasticity tests; chemical analysis; immersion-compression tests of bituminous concrete mixtures; and softening point tests of asphalt-filler mortars. Emphasis in this report has been placed upon the use of infrared spectroscopy for identifying the clay-like nature of the filler.

Introduction

BECAUSE OF the difficulties related to evaluating the suitability of mineral fillers for bituminous mixtures solely on the basis of their physical characteristics, the Bureau of Public Roads has been conducting research to determine the usefulness of new instrumental means for determining the mineralogical nature of fillers—such information to be used as a supplement to that obtained from physical tests. Bituminous technologists are well-aware of the fact that the properties and performance of a bituminous mixture can be influenced greatly by the fraction of aggregate that passes the 200 mesh sieve. It is also known that the effect of this fraction, generally referred to as the filler, on a bituminous mixture cannot be predicted precisely from the easily measured characteristics of the filler itself. The difficulties encountered and some of the relationships between various laboratory tests have been discussed by Warden, Hudson, and Rowell (*1*) [2] in a recent evaluation of a number of filler materials. A general classification of materials on the basis of several tests has been provided in their work but, as they have pointed out, it is impossible to evaluate properly the suitability of a filler on the basis of its physical characteristics alone.

The evaluation of fillers, by noting their behavior either in combination with asphalts or in bituminous concrete mixtures, is made complex by the effect of variations in the test specimens or differences in the properties of the supplementary materials. For example, the immersion-compression test (*2*) generally is used to determine the effect of water on composite specimens of aggregate, filler, and binder. Various laboratories, however, sometimes reach different conclusions as to the effect of a specific filler material because of variations in the type and grade of the aggregate and/or the proportion and characteristics of the asphalt used in the mixture.

The primary objective of the study conducted by Public Roads was to evaluate the potential suitability of a specific deposit of finely divided natural mineral proposed for use as a filler in bituminous concrete. An uncertain classification of the minerals present in this material had been obtained by others from limited chemical and petrographic examinations. In particular, a question existed as to whether kaolinite or other clay minerals were present in this material. The use of conventional procedures and various special techniques used to evaluate this material are discussed in this article. It is believed that the experience reported here, particularly as to the special techniques, may be helpful as a guide to others confronted with a similar problem. Techniques used during this research included x-ray diffraction, differential thermal analysis, and infrared spectroscopy.[3] Special emphasis has been placed upon the use made of infrared spectroscopy to identify the clay-like nature of the filler material from this deposit.

The studies made with each of the various techniques used in the analysis of the material from the deposit were carried out more or less independently, and data resulting from the use of each of the techniques is presented separately. Because the use of infrared spectroscopy as a means of evaluating the mineral characteristics of filler material is relatively new, details on the application of this technique and the manner of interpreting the results have been presented. As literature is readily available concerning procedures and techniques for x-ray diffraction, differential thermal analysis, and chemical analysis, detailed descriptions have not been included for these applications.

Gradation, General Properties, and Classification of Material

Eight samples, representative of different areas of the deposit, were taken directly from the ground and from material previously obtained from the deposit and stored in stockpiles and storage bins. Results of mechanical analysis, surface area measurements, and soil classifications are given in

[1] Presented at the 41st annual meeting, Highway Research Board, Washington, D.C., January 1962.

[2] References indicated by italic numbers in parentheses are listed on page 52.

[3] The authors extend their appreciation to E. B. Kinter and S. J. Diamond, Highway Research Engineers, Bureau of Public Roads, who interpreted the data relating to differential thermal analysis, surface area, and x-ray analysis.

Table 1.—Physical properties of mineral fillers

	Filler material from test deposit, sample numbers—								Control fillers	
	1	2	3	4	5	6	7	8	Limestone dust	Granite dust
Grading:										
Filler passing sieve:										
No. 10 ... percent	100	100	------	100	------	100	100	100	------	------
No. 40 ... do	99	99	------	99	------	99	99	99	100	------
No. 60 ... do	99	99	100	99	100	99	98	99	99	100
No. 200 ... do	95	95	97	97	98	96	96	97	89	97
Filler smaller than (mm):										
0.050 ... percent	92	93	95	95	96	94	95	95	73	62
.020 ... do	70	70	72	73	78	72	77	77	27	22
.005 ... do	26	24	26	28	34	26	37	30	8	5
.002 ... do	2	2	2	2	6	6	11	4	5	1
.001 ... do	0	0	0	0	1	2	3	1	4	0
Surface area:										
Square meters per gram [1]	7	7	7	7	[2] 18	7	[2] 18	7	0.2	0.1
Liquid limit	33	31	34	34	48	34	39	34	------	------
Plasticity index	7	5	6	7	11	6	10	7	[3] NP	[3] NP
AASHO classification	A-4(8)	A-4(8)	A-4(8)	A-4(8)	A-7-5 (10)	A-4(8)	A-4(8)	A-4(8)	A-4(8)	A-4(8)

[1] Approximations obtained by gylcerol retention method.
[2] Comparatively high values are believed to have been caused by greater amount of vermiculite present in samples 5 and 7.
[3] Nonplastic.

table 1. Also shown in the table are characteristics of the two control fillers that were used in subsequent evaluation tests.

All eight samples of the natural filler material had plastic properties; the plasticity indices varied from 5 to 11 and the liquid limits from 31 to 48. With the exception of samples 5 and 7, surface areas as determined by the glycerol-retention method were 7 square meters per gram. This value was less than might be obtained for a pure clay such as kaolinite, approximately 20 square meters per gram, but was substantially higher than the values for the control fillers of limestone and granite, which had surface areas respectively of 0.2 and 0.1 square meter per gram, as can be seen in table 1.

On the basis of these conventional tests as well as infrared spectroscopy and x-ray analysis, which will be discussed later, three samples—2, 5, and 8—considered as representing the maximum differences in composition of the various areas of the deposit were selected for special studies in combination with asphalt. The effect of the natural filler on the properties of bituminous mixtures was determined by the immersion-compression test. The effect of these samples on the softening point of filler-bitumen mortars also was determined.

Table 2.—Immersion-compression test results
(Each value shown is the average for three specimens)

Characteristics determined	Specimens containing—				
	Limestone dust	Granite dust	Filler material from samples—		
			No. 2	No. 5	No. 8
Bulk specific gravity	2.244	2.255	2.281	2.277	2.281
Void characteristics, percentage:					
Air voids	8.8	8.5	6.9	6.9	7.3
Mineral voids	23.1	22.8	21.4	21.3	21.8
Mineral voids filled	61.8	62.6	67.6	67.6	66.5
Compressive strength data, p.s.i.					
No immersion	296	302	347	346	342
After immersion at 120° F. for:					
4 days	230	241	191	207	217
7 days	213	217	138	163	185
14 days	196	198	100	113	121
Retained strength, percentage:					
No immersion	100	100	100	100	100
After immersion at 120° F. for:					
4 days	78	80	55	60	64
7 days	72	72	40	47	54
14 days	66	66	29	33	38
Swell, percentage:					
After immersion at 120° F. for:					
4 days	1.0	1.0	2.2	1.8	1.7
7 days	1.5	1.5	3.4	2.7	2.2
14 days	1.4	1.5	4.8	3.8	3.3

Immersion-Compression Tests

Immersion-compression tests were mad with the three selected samples to determin the effect of the natural filler materials on th strength properties of a bituminous concret mixture. For comparison, the same test were made with two control fillers: a lime stone dust and dust from a crushed granit aggregate. The gradation of the combine aggregate and filler used in the bituminou mixture for the immersion-compression te conformed to that of a typical wearing cour mixture, as follows:

Sieve size	*Percent passing*
½ inch	100.0
⅜ inch	98.5
No. 4	85.0
No. 10	67.0
No. 40	33.0
No. 80	15.0
No. 200-Pan (mineral filler)	6.5

Each of the fillers was sieved to remove material retained on the 200 mesh sieve a the portion passing this sieve was used to co stitute the entire fraction shown as N 200-Pan.

Each mixture contained 6.50 percent a phalt on a total mix basis. Characterist of the asphalt were: Specific gravity, 1.01 softening point, 118° F.; penetration (1 grams, 5 seconds at 77° F.), 93; ductility centimeters/minute at 77° F.), 230 cen meters.

Bituminous concrete mixtures were p pared with filler material from each of the th samples (2, 5, and 8) and each of the t control samples. The aggregate for each m ture, including the filler, was blended a heated overnight at a temperature of 325° and, prior to mixing, the asphalt was heat to a temperature of 290° F. The hot gregate and asphalt were mixed together 2 minutes in a modified Hobart mechani mixer and the mixture immediately s molded into cylindrical specimens by e double-plunger method at 3,000 p.s.i. held 2 minutes. Twelve specimens, 3 inches h and 3 inches in diameter, were molded w each of the five fillers—a total of 60 specim These specimens were cooled to room te perature and then placed in an oven at a te perature of 140° F. to cure for 24 hours.

The 12 specimens of each mixture w divided into 4 groups so that the average b k specific gravity of each group of 3 specim was the same. These groups were then te d for compressive strength at 77° F., one gr p without having been immersed, and each o the other groups after having been immer d in water at 120° F. for 4, 7, and 14 days, e spectively. Average test values for each o the groups are shown in table 2. Results s are shown for void characteristics, comp s sive strength prior to immersion, and c n pressive strength and percent of swell a er immersion.

The molded specimens containing the n u ral fillers, samples 2, 5, and 8, had lo er

percentages of air voids and higher percentages of mineral voids filled with asphalt than the specimens containing limestone and granite dust fillers. Without immersion, the compressive strengths of the specimens containing natural fillers were higher than those of the specimens containing the control fillers. However, after immersion the strength of the specimens with the natural fillers decreased at a much higher rate. The mixtures prepared with the natural fillers would not be acceptable under the criterion used by the Bureau of Public Roads, which requires a minimum of 70 percent retained strength after 4 days of immersion in water at a temperature of 120° F. The mixtures containing the limestone and granite dust would be considered satisfactory. Both the high percentage of swell and the large loss of compressive strength of the specimens containing natural fillers indicate that materials of this type are water-susceptible and may not be satisfactory for highway construction.

Softening Point of Asphalt-Filler Blends

Previous studies, reported in references *1*, *3*, and *4*, have shown that the properties of blends of filler and asphalt are indicative of the effectiveness of mineral fillers. To compare the natural fillers and the two control fillers used in this study, various blends of asphalt and filler were prepared and their softening points determined by the Ring and Ball method. The results of these tests are given in table 3, and the relations of the volumetric filler-bitumen ratios to the softening points of the different blends are shown in figure 1.

Blends prepared from samples 2, 5, and 8 showed a much higher rate of increase in softening point than the blends with the two control fillers. This can be attributed partially to the higher surface area of the natural fillers. Blends containing sample 5 showed the greatest increase in softening point, and the curve for these blends is similar to a curve developed for kaolin by Warden (*1*) and others. A moderate increase in softening point is desirable for increased stability but large increases are considered detrimental because they may result in brittleness of asphalt-aggregate paving mixtures.

Differential Thermal Analysis and X-Ray Diffraction

The combined results of differential thermal analysis and x-ray diffraction on the eight samples from the natural deposit are given in table 4. The results were conclusive with respect to the qualitative presence of kaolinite, quartz, and mica. The quantitative approximations of the amount of kaolinite were made by differential thermal analysis, and the estimates for quartz were obtained by x-ray diffraction. These estimates were made to the nearest 5 percent, which for these minerals is the limit of precision for the techniques used. Although the x-ray patterns very definitely indicated the presence of mica, quantitative estimates could not be made with these techniques. Consequently, mica was estimated by difference. Therefore, any other minerals or amorphous material that may have been present were included with the amount shown for mica, in table 4.

As shown in table 4, the composition of the eight samples from different locations in the deposit was not uniform. The percentage of kaolinite varied from 10 to 45, that of quartz from 10 to 30, and the balance, which included the mica, varied from 45 to 65 percent. Although not shown as a major constituent in table 4, the presence of vermiculite was detected in all samples; 0.7 percent was estimated to be present in all samples except 5 and 7, and

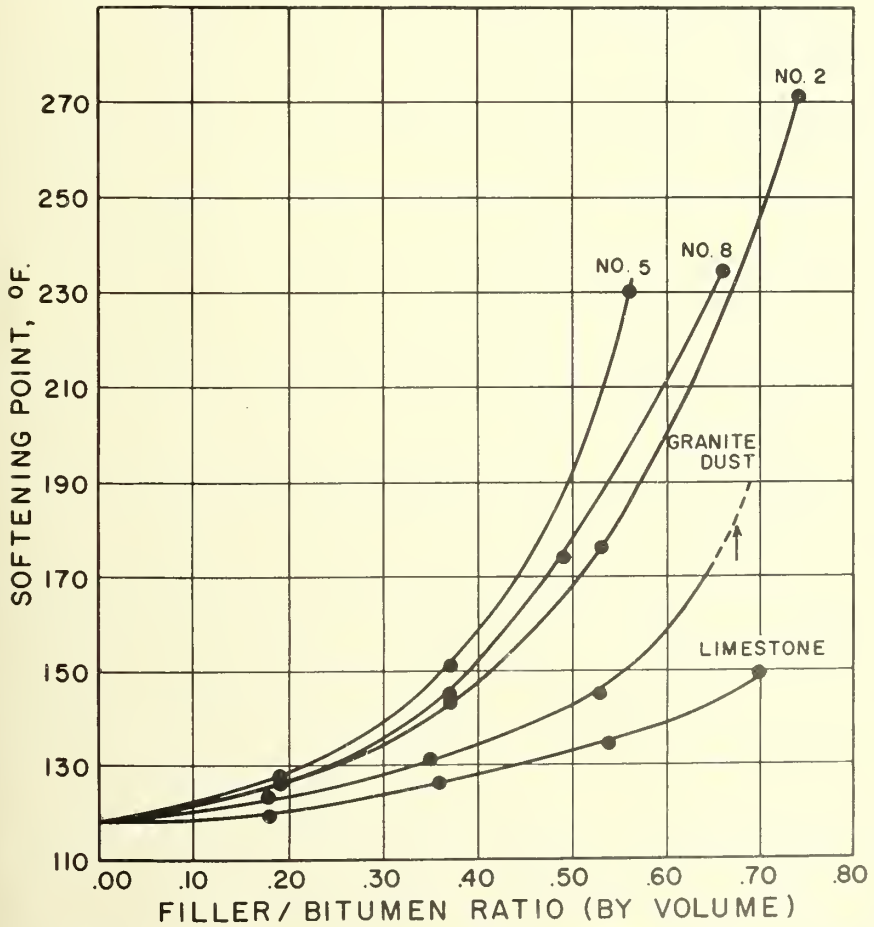

Figure 1.—Relation of filler material to variation in softening point of filler-bitumen mortars.

Table 3.—Softening points of filler-asphalt blends [1]

Filler		Filler-asphalt blends		
Test material	Specific gravity	Filler-bitumen ratio—		Softening point, ° F.
		By volume	By weight	
Limestone dust	2.71	0.18	0.49	119
		.36	.96	126
		.54	1.44	134
		.70	1.97	149
Granite dust	2.79	.18	.49	123
		.35	.97	131
		.53	1.44	145
		.68	1.87	[2] 180+
Sample 2	2.70	.19	.50	126
		.37	.99	143
		.53	1.40	176
		.74	1.97	[3] 271
Sample 5	2.69	.19	.49	127
		.37	.93	151
		.56	1.47	[3] 230
Sample 8	2.70	.19	.50	126
		.37	.99	147
		.49	1.30	174
		.66	1.75	[3] 234

[1] Softening point of the asphalt with no filler was 118° F. and all filler material used in the blends had been dry screened so that 100 percent passed the 200 mesh sieve.
[2] Sample fell from test mold at 180° F.
[3] Glycerine bath.

Table 4.—Mineral content of filler determined by differential thermal analysis and x-ray diffraction [1]

Sample number	Mineral content—		
	Kaolinite [2]	Quartz [3]	Balance (including mica) [4]
	Percent	*Percent*	*Percent*
1	15	25	60
2	10	30	60
3	15	20	65
4	20	20	60
5	45	10	45
6	25	25	50
7	20	20	60
8	15	25	60

[1] Results estimated to the nearest 5 percent.
[2] Estimated from differential thermal analysis.
[3] Estimated from x-ray diffraction patterns.
[4] The presence of mica was established by x-ray diffraction. Approximately 0.7 percent of vermiculite was present except for samples 5 and 7 with approximately 1.5 percent. Amorphous materials that may have been present also are included in the percentages shown.

it was estimated that they contained 1.5 percent. Because of the very large surface area of vermiculite, differences of such magnitude in the vermiculite content of a filler are significant and, in this case, accounted for the differences in the surface areas previously noted for samples 5 and 7 (table 1). The high liquid limits and plastic indices for these two samples also can be accounted for on the basis of vermiculite content.

Chemical Analysis

Results from the limited chemical analyses of the eight samples of natural filler material are shown in table 5. These results tended to confirm the more specific findings from the x-ray analysis. The substantial ignition loss that was noted is indicative of combined water, such as is found in clay minerals. Although by no means conclusive evidence, the appreciable potassium oxide content, as compared with very little sodium oxide, suggested that the bulk of the mica found by x-ray diffraction was of the muscovite type. The estimate of the amount of muscovite mica present was based therefore on an empirical calulation from the potassium oxide content. The higher ignition loss for sample 5 confirmed the x-ray diffraction findings that this sample had the largest clay content of any of the eight samples.

Table 5.—Chemical analysis of natural filler material

Sample number	Moisture loss at 105° C.	Loss on ignition between 105° and 950° C.	Potassium oxide, K_2O [1]	Sodium oxide, Na_2O [1]	Muscovite mica, by empirical calculation [2]
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
1	0.12	3.8	3.23	0.20	40
2	.10	3.2	3.13	.20	35
3	.08	3.8	3.13	.20	35
4	.15	3.8	3.18	.23	35
5	.47	8.1	2.16	.28	25
6	.22	5.9	2.49	.25	30
7	.53	5.2	2.61	.19	30
8	.10	3.6	2.93	.20	35

[1] Calculated on oven-dry basis.
[2] Computed to nearest 5 percent from potassium oxide content by assuming that muscovite mica contains an average of 8.5 percent (*6*).

INFRARED SPECTRAL ANALYSIS

As previously noted, the application of infrared spectroscopy to a problem involving the identification of minerals in natural filler material is relatively new. Therefore, the procedures used in this portion of the study are described in greater detail, and the results are discussed with reference to the findings from more familiar techniques.

Instrument and Its Operation

A double-beam spectrophotometer with automatic recording, as described in a previous report (*5*), and having a sodium chloride prism for operation at wavelengths of between 2.5 and 15 microns, was used for this study. Complete scanning time was approximately 12 minutes. The sample was prepared for analysis by the potassium bromide (KBr) disk method. This method involves dispersing a minute amount of test sample in powdered potassium bromide, pressing the mixture into a disk, and recording the transmitted infrared spectrum of the disk. Because KBr does not absorb significant infrared energy in the wavelength range used for this study, the resultant spectra are characteristic of the test samples alone. Details of this procedure are described in the following paragraphs.

Samples of filler material were ground to a fine powder with mortar and pestle and dried at a temperature of 105° C. for at least 24 hours. Weighed samples of approximately 1 milligram were added to 0.35 gram of KBr (anhydrous spectroscopic grade, 200/325 mesh) and placed in a stainless steel capsule. Two stainless steel balls were added, and the contents were mixed for 30 seconds in an electric, dental-type amalgamator. The mixed powder was transferred to a special die and the assembly was evacuated to a pressure of less than 1 centimeter of mercury. While the vacuum was maintained, a total load of 20,000 pounds was applied to the die for several minutes. The pressed disk was removed from the die with tweezers and analyzed in the infrared spectrophotometer. Each disk measured 13 millimeters in diameter, 1 millimeter in thickness, and had a fairly transparent appearance.

Spectra

The infrared spectrum recorded for each of the eight samples has been reproduced in figure 2. (The spectra shown in figures 2–4 are included in this article only for illustrative purposes; detail has been lost in reproduction processes.) The spectra had characteristics attributable to materials with a layered silicate structure similar to clay, and they also revealed the presence of quartz. One of the clay minerals could be identified clearly as kaolinite from a comparative study of published spectra of clay minerals. To confirm this, a comparison of pertinent reference spectra of kaolinite, quartz, and muscovite is shown in figure 3. The spectrum for quartz characteristically has a prominent double peak (downward dip) between 12.5 and 12.8 microns. Kaolinite has an interesting spectrum that includes a sharp peak at 9.0, a doublet or double peak at 9.65 to 9.9, and another peak at 10.9 microns. The kaolinite spectrum is characterized also by a slight shoulder on the low wavelength side of the latter peak, which occurs at about 10.6 microns. The spectrum for muscovite is characterized by major peaks at about 10.0 and 10.9 microns. The spectra for both kaolinite and muscovite are characteristic of the layered silicates, that is, clay minerals.

The spectra for both kaolinite and muscovite in this study showed double peaks in the 2.7 to 3.0 micron area, which is characteristic of clay structures. The 2.7 micron peak was produced by unbonded hydroxyl (OH), and the 2.9 peak was caused by bonded hydroxyl. In these spectra the 2.9 peak was somewhat enhanced by the effect of the KBr powder itself, which contained unremovable moisture; this was evident also in the spectrum for quartz where the 2.9 peak was caused by moisture in the KBr pellet.

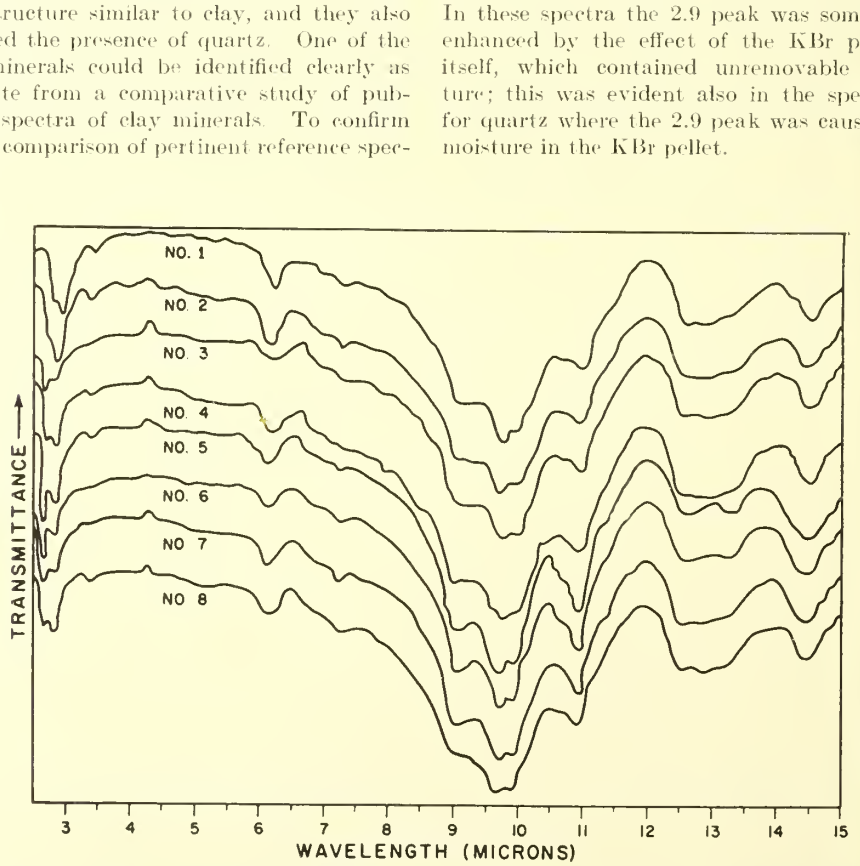

Figure 2.—Infrared spectra of eight samples of material from deposit under test.

For a more direct comparison and evaluation, the spectrum of the filler in sample 5 was replotted, as shown in figure 3, because it exhibited the closest similarity to the kaolinite spectrum. From this comparison the following conclusions were drawn and are applicable in varying degrees to the material in the other seven samples.

Quartz

The presence of quartz in sample 5 was apparent from the prominent peak in the spectrum between 12.5 and 12.8 microns. The true doublet formation in this area, which is characteristic of quartz, was largely attenuated in the sample 5 spectrum because of the background effect of other materials.

Kaolinite

The presence of kaolinite in sample 5 was evident from the prominent peak in the spectrum at 9.0, the doublet at 9.65 to 9.9, and the peak at 10.9 microns. The slight shoulder at 10.6 microns also confirmed the presence of kaolinite. The peak at 9.0 microns was not as sharp as in the kaolinite spectrum, perhaps as a result of the additive background effects of quartz and other substances in the sample filler material. The kaolinite pattern was most pronounced in the spectra for samples 5, 6, and 7 (fig. 2) and this is in conformity with the differential thermal analysis and loss on ignition data, as can be seen by referring to tables 4 and 5.

Muscovite

Because all the major infrared peaks of muscovite appear at wavelengths common to some of the kaolinite peaks, it was impossible to confirm the presence of muscovite in the filler material by infrared spectroscopy alone. The presence of muscovite was inferred, however, from the combination of the chemical data—potassium oxide determination—and the slope and slight bulge in the infrared pattern in the 10.2 micron area. This infrared characteristic was slightly more obvious in the spectra for the other materials than for that from sample 5. Nevertheless, on the basis of the infrared data alone, it cannot be said with certainty that muscovite was present.

Presence of Clay Confirmed

Because the clay mineral structure is destroyed by ignition, the use of infrared spectroscopy was explored to provide additional pictorial confirmation of the presence of clay minerals by examination of spectra from the filler material after it had been ignited. It is known that, specifically in the case of kaolinite, ignition at temperatures between 500° and 900° C. produces a metakaolin and, consequently, the 10.9 micron peak in the spectrum is destroyed because of the dislocation of the octahedral sheet. Spectra of ignited kaolinite, ignited kaolinite and muscovite, and ignited material from sample 6 are shown in figure 4. Ignitions were made at a temperature of 600° C. for several hours. By noting the absence of the characteristic peaks in the various spectra—that is, the 2.7 micron peak of unbonded hydroxyl and the 10.9 micron peak of a layered silicate—it can be seen that in all cases the ignition destroyed the clay minerals.

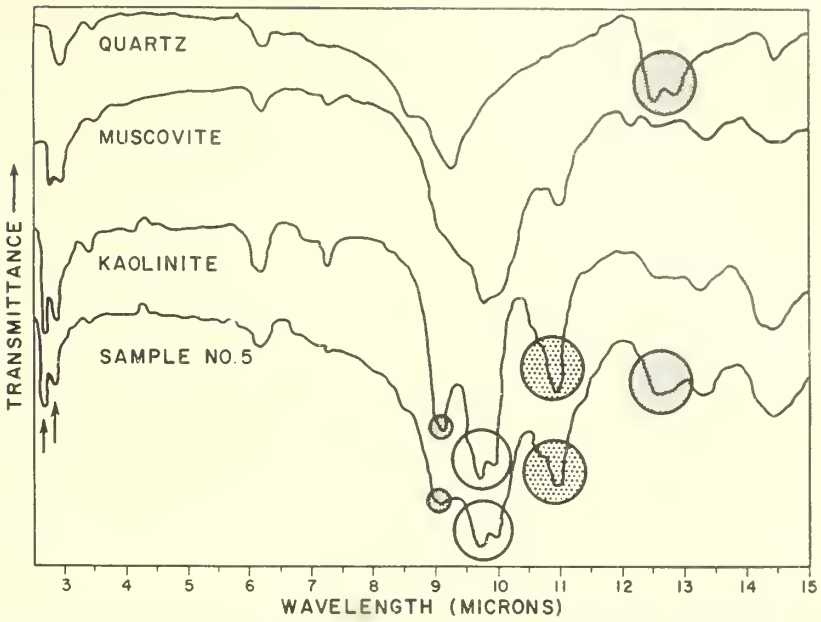

Figure 3.—Comparison of infrared spectra of reference materials with spectrum of sample 5 from the deposit under test.

The evidence from examination of the spectra of these materials after ignition strongly supports the conclusion that significant amounts of clay minerals were present in the original samples of the natural filler material. It was noted that the quartz peak of 12.5 microns in the spectrum of the ignited sample 6 remained unaffected by the ignition. As heating would have little effect on quartz, this would be expected. Note that the spectrum for the ignited sample 6 appears to resemble a composite of the spectrum of ignited kaolinite and muscovite, except for showing the influence of quartz.

Findings Concerning Techniques

On the basis of this study, it is believed that further research is warranted on the application of the techniques of differential thermal analysis, x-ray diffraction, and infrared spectroscopy to problems of the type described in this article. These techniques are expected to be helpful in explaining the varying effects that material passing the 200

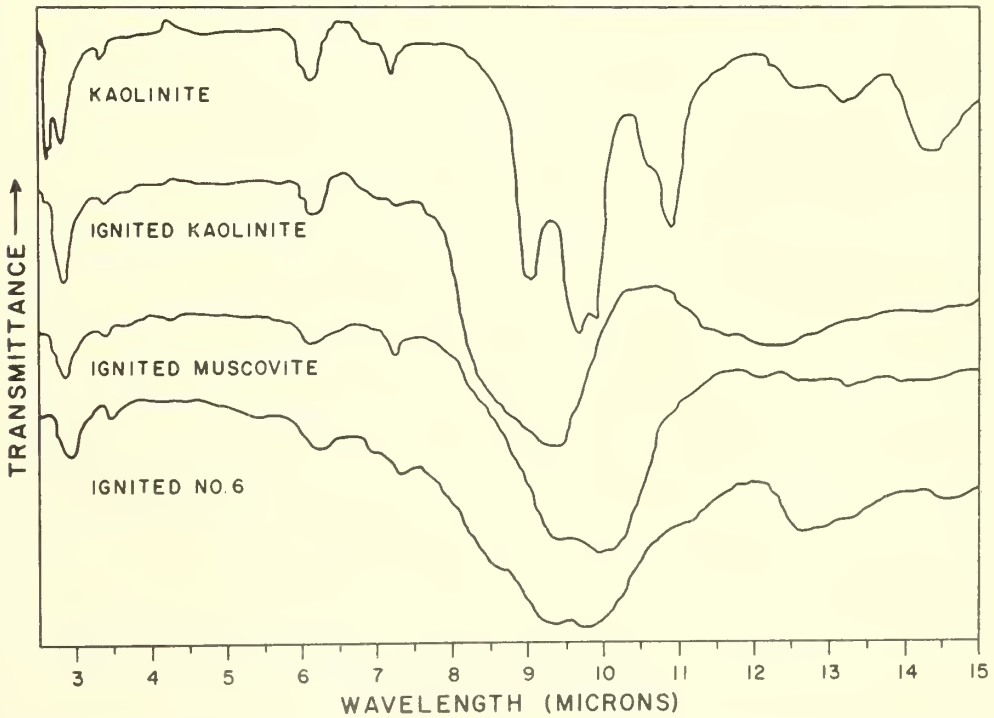

Figure 4.—Comparison of effect of ignition on spectra of minerals and spectrum of sample 6 from the deposit under test.

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Northwest Expressway, Interstate Route 94. This view, west from Augusta Blvd., Chicago, Ill., shows a section of the reversible roadway for six lanes of traffic and the median shoulders for emergency parking.

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Characterization of Montmorillonite Saturated with Short-Chain Amine Cations

Part I—Interpretation of Basal Spacing Measurements

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by SIDNEY DIAMOND
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The clay mineral montmorillonite is widely distributed in soils and has a considerable effect on soil properties such as plasticity, strength, and volume change. Recently, various amines (cationic organic compounds, derivatives of ammonia) have been tried in soil stabilization, usually as supplementary additives with other stabilizers such as phosphoric acid. The following two articles present information on two aspects of the products obtained by treating a montmorillonite (Wyoming bentonite) with a number of these amines. The results presented are not of immediate practical value in application to field problems, but are basic in nature; they represent a part of a larger study investigating the nature and properties of soil clay minerals and their contribution to the engineering performance of soil materials. In the first article the data concern measurements of the basal spacings of the amine-saturated montmorillonite.

Basal spacing measurements were made by x-ray diffraction. Specimens prepared by saturating portions of the montmorillonite with a series of small aliphatic primary, secondary, and tertiary amine cations, and with quaternary ammonium cations were x-rayed while wet, after oven-drying, and after glycerol treatment. Observed basal spacings ranged from about 12 to 14 A., depending on the cation, indicating that a monolayer of cations always was interleaved between adjacent montmorillonite layers. The observed spacings are consistent with the concept that the cations were oriented with their minimum thickness in the c-axis direction, but it is possible that the smallest cations had a long axis in this direction and were partially embedded in the clay surfaces.

Montmorillonite saturated with cations containing up to four carbon atoms retained some sensitivity to water, as indicated by a slight contraction of the lattice on oven-drying and by rapid re-expansion on exposure to the humidity of the laboratory atmosphere.

For cations having a layer thickness less than glycerol, limited lattice expansion occurred when specimens were glycerol treated, usually to a spacing sufficient to accommodate a single layer of glycerol molecules. Apparently the cations functioned as pillars separating the montmorillonite layers, and the glycerol molecules expanded the lattice as necessary to enter the spaces between the pillars. However, two exceptions were noted: with methylamine-saturation both a one-layer and a two-layer complex of glycerol were present simultaneously in the same specimen; but in trimethylamine saturation two different, one-layer complexes were present. It is suggested that the glycerol in one-layer montmorillonite complexes may exist in two orientations, one causing a layer thickness of about 4.1 A., the other a thickness of about 4.6 A.

Introduction

THE PROPERTIES of montmorillonite saturated with large aliphatic amine cations have been studied fairly extensively in recent years, but relatively little work has been done with the smaller amine and quaternary ammonium cations. The present work represents an attempt to characterize the complexes formed on saturation of a montmorillonite with some of these smaller cations, particularly as to cation dimensions, packing of ions on the clay surface, and complexes formed by glycerol. This article includes information on the determination of basal spacings of the various complexes, and on the interpretation of the data in terms of the thickness and orientation of the cations and of the glycerol molecules.

Summary and Conclusions

It has been shown that montmorillonite saturated with any of a number of short-chain normal primary, secondary, or tertiary amine, or quaternary ammonium cations acquires a basal spacing characteristic of the formation of a complex consisting of a monolayer of cation pillars interleaved between adjacent layers of montmorillonite. The basal spacing is generally very regular, even with wet specimens, and the spacing is not significantly reduced by air-drying. Oven-drying causes slight contractions in spacing ranging from about 0.7 A. (angstrom unit, 10^{-8} cm.) for specimens saturated with the trimethylamine cation down to about 0.1 A. for those saturated with cations containing six or more carbon atoms. Upon re-exposure to the laboratory atmosphere, specimens that showed significant contractions on oven-drying underwent a rapid, partial re-expansion because of the sorption of water vapor.

Thicknesses of the cation layers estimated from basal spacings of the saturated clay were 0.2 to 1.1 A. smaller than the minimum clearance thicknesses measured from atom models. One- and two-carbon amine cations might have been present in other than minimum thickness orientation with respect to the basal plane and, if so, were partially embedded in the silica sheets.

The effect of glycerol treatment of the clay varied somewhat with the cation. For saturation with most of the smaller cations, basal spacings were obtained that are characteristic of the clearance thickness of a monolayer of glycerol molecules between the montmorillonite layers. However, with saturation by methylamine cations, at least part of the clay expanded to admit two molecular layers of glycerol. This may have been a reflection of the inhomogeneity of the Wyoming bentonite material, the fully-expanding portion perhaps having a lower layer-charge density than the remainder. With trimethylamine-treated clay, two distinct, one-layer complexes of glycerol occurred simultaneously in the same specimen. This also may have been an indication of the inhomogeneity of the montmorillonite.

In general, it is clear that with saturation by the smaller amine cations, glycerol penetrated the lattice and a complex consisting of

[1] Presented at the 10th National Conference on Clays and Clay Minerals, Austin, Texas, October 1961.

[illegible] glycerol molecules were formed in the interlayer spaces, with the separation between adjacent montmorillonite layers being controlled by the glycerol molecules. Generally, the orientation of the glycerol molecules apparently corresponded to that observed in the normal two-layer glycerol complexes of montmorillonite, which had a Δ value of about 4.15 A. per layer. Of the two one-layer glycerol complexes with trimethylamine-saturated clays, one glycerol layer had a 4.2 A. Δ-value and the other a Δ-value of 1.6 A. This apparently indicated two different orientations of the glycerol molecules. The 1.6 A. Δ-value indicated an orientation corresponding to that for the one-layer glycerol complex for a sodium-saturated montmorillonite. With the larger tertiary amine cation and the quaternary ammonium cations, the changes in basal spacing observed for glycerol-treated specimens were minimal, and it was not possible to decide unequivocally whether the lattice actually had been penetrated by glycerol molecules.

Materials and Experimental Procedure

Volclay-brand Wyoming bentonite was used as a source of montmorillonite. The commercial product was fractionated by dispersion in water with sodium hexametaphosphate (Calgon) and sedimentation by centrifugation; the material coarser than 2-microns was discarded.

Amine cation-saturated specimens of the clay were prepared for x-ray diffraction by using 1-normal solutions of amine compounds as follows: methyl-, ethyl-, dimethyl-, diethyl-, di-n-propyl-, trimethyl-, triethyl-, tri-n-propyl-, and tri-n-butylamine hydrochloride. Saturations also were made with solutions of two salts, tetramethylammonium chloride and tetraethylammonium bromide. For each specimen an aliquot of the clay-water dispersion sufficient to yield about 100 milligrams of clay was used to prepare a thin film of clay as an oriented aggregate on a porous ceramic tile, by a centrifuge method (*1*).[2] Saturation was accomplished by passing five separate, 5-milliliter portions of the amine salt solution (50 times the exchange capacity) through the clay film and tile held in the suction device described in reference *1*. The clay film was then washed five times by passing 20-milliliter portions of distilled water through the film and tile in the same manner.

The solutions of the primary, secondary, and tertiary amine hydrochlorides were prepared by neutralizing the appropriate amine with concentrated hydrochloric acid and diluting to 1-normal with distilled water. Corresponding solutions containing the quaternary amine cations were prepared directly from the two salts, tetramethylammonium chloride and tetraethylammonium bromide. The amines and quaternary ammonium salts were Eastman organic reagents.

X-ray diffraction patterns were recorded with a General Electric XRD 3 diffractometer employing copper Kα radiation. These patterns were obtained for specimens when they were (1) wet, directly after the saturation and washing treatment; (2) air-dried; (3) oven-dried overnight at 110° C.; and (4) glycerol-treated.

For the glycerol treatment, an excess amount of glycerol was applied to the clay film; the specimen was placed over a supply of glycerol in a sealed container and tempered overnight at 70° C. After this tempering treatment, the specimens had substantially the same basal spacings as those treated with glycerol alone, but the tempering markedly improved the regularity of the sequence of basal reflections and greatly increased the intensity of the individual maxima.

Results and Discussion

Wet specimens

Basal spacings of the wet amine-cation saturated specimens are listed in table 1. The numbers given are averages calculated from several higher orders of reflection, and they occupy a restricted range of 12.6 to 14.2 A. For most specimens, the x-ray maxima were surprisingly sharp and well-defined, and they had a very regular sequence of reflections. This was in contrast to the x-ray maxima for wet specimens of the same montmorillonite saturated with inorganic cations; such specimens commonly exhibited only diffuse reflections lacking in higher orders. As the basal spacing of the montmorillonite structure itself was 9.4 A. and the thickness of a methyl group (*2*) is about 4 A., it is clear from the basal spacings, as shown in table 1, that no more than a monolayer of cations was present between adjacent layers of montmorillonite.

Air-dried specimens

Small reductions in the basal spacings of about 0.1 A. were caused by the air-drying of the wet amine-cation saturated specimens. The basal spacing data for the air-dried specimens were not included in table 1 because the effect of air-drying was so slight and the relative humidity had not been controlled.

Table 1.—Basal spacings of wet- and oven-dry amine-saturated montmorillonite

Cation	Basal spacings		
	Wet specimens	Oven-dried specimens	Difference
	A.	A.	A.
Methylamine	12.6	[1] 12.16	0.4
Ethylamine	12.94	[1] 12.66	0.3
Dimethylamine	12.94	[1] 12.52	0.4
Diethylamine	13.22	[1] 12.81	0.4
Di-n-propylamine	13.28	13.15	0.1
Trimethylamine	13.5	[1] 12.84	0.7
Triethylamine	13.40	13.26	0.1
Tri-n-propylamine	13.4	13.36	0
Tri-n-butylamine	13.6	13.51	0.1
Tetramethylamine	[2] 14	13.85	([2])
Tetraethylamine	14.2	14.17	0

[1] Averages of estimates based on higher orders of reflection, each order of reflection having been recorded individually immediately after a separate drying period.

[2] Diffuse nature of x-ray pattern of the second and higher orders precluded accurate estimation of basal spacing and of difference in spacing between wet- and oven-dry specimens

Oven-dried specimens

The basal spacings for the oven-dried specimens also are given in table 1. In the normal x-raying procedure, an oven-dried specimen was removed from the oven, mounted directly on the diffractometer, and the appropriate two-theta range (2° to as high as 60°) was covered in a continuous scan. This technique was satisfactory for specimens saturated with the larger amine cations; but, for those with the smaller cations, rapid adsorption of water vapor from the atmosphere caused considerable expansion of the lattice to take place while the diffraction patterns were being recorded. As appropriate accessory equipment was not available to prevent this adsorption and expansion during the x-raying, a special procedure to minimize the effect was adopted to record the patterns. The diffractometer was preset to within one-half a degree of the expected position for a given maximum, the specimen was removed from the oven and mounted immediately in the diffractometer, and the maximum was recorded. The specimen was then returned to the oven to remove the small amount of moisture that had been adsorbed during the recording period, and the procedure was repeated to record the next order of reflection. For each recording, approximately 1½ minutes elapsed between the removal of the specimen from the oven and the recording of the maximum; the specimen remained hot during this period.

The basal spacings with this procedure were estimated from four or five individual higher orders of reflection, which agreed within 0.05 A. for most specimens and within 0.10 A. for all. The regularity of spacing was comparable to that observed for specimens saturated with the larger cations, by use of the normal continuous scan procedure.

As shown in table 1, for clay specimens saturated with the smaller amine cations (up to four carbon atoms), the basal spacings were not fixed; the spacings for the wet specimens were from 0.3 to 0.7 A. higher than those for oven-dried specimens. It was clear that despite the amine saturation, water penetrated the interlayer spaces of specimens and slightly expanded the lattice. But when specimens were saturated with cations containing six or more carbon atoms, the result was virtually fixed basal spacings. The differences in spacings between wet and oven-dry specimens were not more than about 0.1 A.; these differences are consistent in direction and probably are not much greater than the experimental error.

Jordan (*3*) has shown that in the series of primary normal aliphatic amine cations those up to decylamine yield one-layer complexes with montmorillonite, although dodecylamine and larger cations form two-layer complexes. Little information is available in the literature concerning the size of cation at which a corresponding change takes place in the secondary or tertiary amine series. In this study, one-layer complexes were observed in specimens with the largest of the secondary and tertiary amine cations em-

[2] The references indicated by italic numbers in parentheses are listed on page 56.

loyed, di-n-propylamine and tri-n-butylmine. Although no further exploration was ade in the secondary amine series, specimens aturated with the next higher member of he tertiary series, tri-n-amylamine, were xamined and the basal spacing of approxiately 18 A. indicated the presence of a two-ayer complex.

Recently, a two-layer complex of methylmine- or dimethylamine-saturated montorillonite was obtained by treatment of Vyoming bentonite with concentrated soluions of methylamine and dimethylamine ydrochloride (*4*). However, for these mallest two cations only one-layer complexes vere formed when the clay was treated, as in his study, by repeated leaching of a thin film f clay with an excess of 1-normal solution of he hydrochloride. The effect was the same vhether the clay film was x-rayed while still oaked with the amine hydrochloride solution r after thorough washing with distilled water. Presumably, therefore, formation of a two-ayer complex with methylamine or dimethylmine requires that the clay be in contact with solution more concentrated than 1-normal.

Little published information is available on asal spacings that could be used for comparion with the basal spacings observed in this tudy for the oven-dried specimens. Spacings enerally 0.1 to 0.3 A. less than those recorded or this study have been cited (*5*, *6*) for a ontmorillonite that had been saturated with ethylamine, dimethylamine, trimethylamine, etramethylammonium, and tetraethylamnonium cations, all of the specimens having een dried and outgassed under high vacuum. A spacing of 13.3 A. was reported (*7*) for both vet and air-dried specimens of a montorillonite saturated with triethylamine ations, which is comparable to the 13.40 A. nd 13.26 A. obtained in this study for a wet nd an oven-dried specimen.

In discussing the basal spacings of clayrganic complexes, it has been customary to stimate the thickness of the organic layer by ubtracting the theoretical thickness of the ilicate layer (9.4 A. for montmorillonite) rom the basal spacing measured by x-ray liffraction. MacEwan (*8*) called this estimate f the thickness of the organic layer the "Δ-value." Δ-values calculated in this way or the several amine and quaternary ammonium cations used in this study are presented in able 2. For comparison, also included are the orresponding minimum clearance thicknesses or the various cations; these were obtained by irect measurement of Fisher-Taylor-Hirschelder atom models. Each Δ-value is somewhat smaller than the minimum clearance thickness measured from the model; the apparent contraction varied from 0.2 to 1.1 A. Similar effects have been observed for adsorbed rganic molecules, as well as for cations (*9*, *10*, *5*). This apparent contraction of the organic layer has been variously ascribed to hydrogen bonding to the silicate surface, to partial keying into the silicate layer, or simply to compression exerted by the host montmorillonite lattice. Brindley and Hoffman (*11*) have reviewed and discussed this subject in considerable detail.

In the foregoing discussion of the basal spacing and apparent contraction data, it has been assumed that the adsorbed ions were oriented so as to present their minimum thicknesses in the direction perpendicular to the layers. However, another interpretation is likely for the smallest amines; that is, those containing only one or two carbon atoms. In a recent paper, Rowland and Weiss (*4*) have suggested that, when montmorillonite is treated with a relatively dilute solution of methylamine hydrochloride or dimethylamine hydrochloride, the adsorbed amine cations may be oriented with their long axis perpendicular to the silicate layers, the cations being partially embedded in the holes of the hexagonal oxygen rings of the silica sheets. The basal spacings observed in this study for these two cations, and also for the ethylamine cation, are consistent with either orientation—the increased clearance thickness required for these cations in their vertical orientation could have been compensated for by their being partially embedded in the 2.4 A.-deep oxygen-ring cavity. Evidence presented in part II of this article strongly supports the latter interpretation.

Glycerol-treated specimens

Basal spacings for amine-saturated montmorillonite specimens treated with glycerol are given in table 3. These spacings give no indication of the formation of two-layer complexes, except for specimens saturated with methylamine. X-ray diffraction patterns of specimens saturated with the methylamine cation and then treated with glycerol disclosed that both one- and two-layer complexes were present simultaneously. This simultaneous occurrence of the two types of complexes may possibly have been caused by the inhomogeneity of the bentonite with respect to layer charge density—that portion of the material with lower charge density tending to expand more readily than that with higher charge density, thus admitting two molecular layers of glycerol, but the higher charge density material tended to admit only one molecular layer. Beavers and Larsen (*12*) showed that a clay fraction of a Wyoming bentonite, analyzed electrophoretically by schlieren moving boundary procedures, behaved as though materials of two or more different charge densities were present.

The patterns of the glycerol-treated trimethylamine-saturated clay, although not showing the presence of a two-layer glycerol complex, were unusual in that they showed two distinct series of reflections and thereby indicated the simultaneous presence of two, different one-layer complexes—a predominant one having a basal spacing very close to 14.0 A., and another, representing only a small part of the clay, having a spacing of about 13.6 A. This variation also might be accounted for by inhomogeneity of the montmorillonite with respect to layer charge density. The x-ray diffraction patterns for all of the other cations showed only a single sequence of strong and well-ordered basal reflections, all for one-layer complexes. The basal spacings cited in table 3 are averages

Table 2.—Apparent contraction of the cation layer in amine-saturated montmorillonite

Cation	Δ-value [1]	Minimum clearance layer thickness derived from model	Apparent contraction
	A.	A.	A.
Methylamine	2.8	3.7	0.9
Ethylamine	3.3	3.9	0.6
Dimethylamine	3.1	3.9	0.8
Diethylamine	3.4	3.9	0.5
Di-n-propylamine	3.7	3.9	0.2
Trimethylamine	3.4	4.5	1.1
Triethylamine	3.9	4.7	0.8
Tri-n-propylamine	4.0	4.8	0.8
Tri-n-butylamine	4.1	4.8	0.7
Tetramethylamine	4.5	5.3	0.8
Tetraethylamine	4.8	5.5	0.9

[1] Observed basal spacing of oven-dried specimen minus 9.4 A., the theoretical thickness of the montmorillonite layer.

calculated from four or five individual, higher orders of reflection. Agreement among the basal spacing estimates calculated from these individual orders of reflection is comparable to the agreement among basal spacing estimates for oven-dried specimens. For comparison with the basal spacings of the glycerol-treated specimens, the corresponding spacings for oven-dried specimens also are given in table 3. In general, the basal spacings for the glycerol-treated specimens are higher and vary with the size of the cation. With methylamine, the smallest cation, the spacing of the one-layer complex produced by the glycerol treatment is 13.6 A., which is about 1.4 A. higher than the spacings for the oven-dried specimens. For the other cations, differences between the spacings of the glycerol-treated and the corresponding oven-dried specimens decrease systematically with increasing size of the cation; the difference ranges from 1.2 A. for dimethylamine down to 0.1 A. or less for specimens saturated with cations having more than six carbon atoms. For the tetraethylammonium cation, the spacing of the glycerol-treated specimen is actually lower than that of the oven-dried specimen, by 0.1 A.

Table 3.—Comparison of basal spacings for glycerated and oven-dried specimens of amine-saturated montmorillonite, and Δ-values for the glycerated specimens

Cation	Basal spacings			Δ-value for glycerated specimens
	Glycerated	Oven-dried	Difference	
	A.	A.	A.	A.
Methylamine	13.60 and 18	12.16	1.44 and 6	4.2
Ethylamine	13.65	12.66	0.99	4.2
Dimethylamine	13.74	12.52	1.22	4.3
Diethylamine	13.40	12.81	0.59	4.0
Di-n-propylamine	13.40	13.15	0.25	4.0
Trimethylamine	13.99 and 13.6	12.84	1.16 and 0.8	4.6 and 4.2
Triethylamine	13.36	13.26	0.10	4.0
Tri-n-propylamine	13.47	13.36	0.11	4.1
Tri-n-butylamine	13.55	13.51	0.04	4.1
Tetramethylamine	13.96	13.85	0.11	4.6
Tetraethylamine	14.05	14.17	−0.12	4.6

Δ-values for glycerol-treated specimens also are given in table 3. These specimens may be placed in two groups, one whose Δ-values centered about 3.15 A. and one whose Δ-values centered about 4.6 A. MacEwan (*8*) gave a Δ-value of 8.3 A. for the two-layer glycerol complex, which corresponds to a thickness of 1.15 A. for the glycerol molecule. He also observed a lower Δ-value of 3.8 A. for the one-layer glycerol complex of halloysite. Previously, the authors (*13*) reported that a one-layer glycerol complex of sodium-saturated montmorillonite had been obtained by heating the glycerol-treated clay. The basal spacing was 13.95 A., which corresponds to a Δ-value of 1.55 A. Thus, the Δ-values reported in table 3 are within the range of previously reported estimates derived from glycerol complexes. Examination of an atom model of the glycerol molecule revealed that a large number of possible conformations and orientations of the molecule can yield Δ-values within this range. Two different orientations or conformations of the glycerol molecules are suggested by the two distinct groups of Δ-values, and also by the two separate sequences of basal reflections observed for one of the cations in each of the two groups. The orientation causing a Δ-value of about 1.15 A. apparently corresponds to the state of the glycerol molecules in the normal two-layer glycerol complex; and the orientation causing the 4.6 A. Δ-value apparently corresponds to the orientation of the glycerol molecules in the one-layer glycerol complex of sodium-saturated montmorillonite.

The concept has been established (*5, 6*) that, in montmorillonite saturated with certain small amine cations, the cations act as "pillars" holding the individual montmorillonite layers apart to a spacing approximating the thickness of the individual cation. The pillars do not form a close-packed layer, and the interlayer space remaining between them is available for sorption of either polar or nonpolar gases. Also established was the fact that further expansion of the basal spacing of the amine-saturated montmorillonite can accompany sorption of certain polar gases whose molecules require greater clearance space than that provided by the amine cation pillars.

The results for the sorption of glycerol in this study are consistent with this pillar concept. For the smaller amine cations, penetration of glycerol molecules into the vacant interlayer spaces between the cation pillars apparently was accompanied by a lattice expansion to the clearance required to accommodate the glycerol molecules themselves. A mixed, one-layer complex of glycerol molecules and amine cations thus may be said to have been present. But, for the larger tertiary amine cations, interlayer separations established in the specimens by the cations themselves approached the clearance required to accommodate glycerol molecules. Adsorption of a monolayer of glycerol molecules in the interpillar spaces therefore would require little, if any, expansion of the lattice. As the changes in basal spacings for these cations were minimal (table 3), it is clear that not more than such a monolayer of glycerol molecules could have been adsorbed; but, it also is not possible to decide unequivocally whether penetration of glycerol into the interpillar spaces actually occurred. A similar uncertainty also applies for the quaternary ammonium cations.

Although similar x-ray data for glycerol-treated, triethylammonium-saturated montmorillonite had been interpreted (*14*) to indicate that glycerol molecules do not penetrate between the unit layers, results from this study indicate that such an interpretation is not necessarily correct. Whether any such penetration actually occurs is discussed more fully in part II.

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Characterization of Montmorillonite Saturated with Short-Chain Amine Cations

Part II—Interlayer Surface Coverage by the Amine Cations

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

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A brief explanation of the basis for research with a montmorillonite clay saturated with a series of amine cations is included in the editorial appearing with the first article. In this Part II, the interlayer surface area of the clay that is covered by the amine cations is discussed.

Portions of Wyoming bentonite were saturated with a series of short-chain aliphatic amine and quaternary ammonium cations and the amine content remaining after washing was determined by combustion analysis. When the clay was saturated with amine cations containing one or two carbon atoms, the amine content of the clay was greatly in excess of its cation exchange capacity. This excess was thought to be either amine cations or uncharged amine molecules that had become partially embedded in the oxygen rings of the silica sheet surfaces. When the clay was saturated with the larger, tertiary amine cations, the amine content of the clay was slightly less than its exchange capacity, presumably because of the blocking of some exchange sites by steric interference.

Quantitative determination of the glycerol retained as a monolayer in glycerol-treated specimens of the amine-saturated clays showed that these clays retained considerably less glycerol than the untreated bentonite. The amounts of retained glycerol were employed to provide quantitative estimates of the interlayer clay surface remaining unoccupied between cation pillars. The percentage of unoccupied interlayer surface of the untreated clay ranged from about 75 percent for the clay saturated with methylamine to essentially zero for the clay saturated with the larger, tertiary amine cations.

The validity of the areas calculated from the glycerol retention data was indirectly confirmed by deriving estimates of the cross-section areas of the several cations from the reductions in the amount of glycerol retained. These experimentally-derived cross-section cation areas were in satisfactory agreement with the areas of the cations projected from atom models.

Introduction

IN MONTMORILLONITE saturated with certain short-chain amine and quaternary ammonium cations, it has been shown (*1, 2*)[2] that the amine cations are present largely as an incomplete monolayer between the unit layers of the montmorillonite, and the individual cations act as pillars in preventing collapse of the montmorillonite lattice when the clay is dried. Also, the space between the cation pillars has been demonstrated to be available for the sorption of both polar and nonpolar gases. In an attempt to measure the extent of the interpillar areas by applying the Brunauer-Emmett-Teller (B.E.T.) equation to their observed data for the sorption of gases, Barrer and Reay (*1*) found that the calculated areas were much lower than had been expected from the cation exchange capacity of the clay and the size of the cations. The principal reason adduced for this discrepancy was that the B.E.T. gas-sorption model seemingly did not apply to the unusual sorption conditions prevailing in the sterically restricted areas between the montmorillonite unit layers.

For montmorillonite and related substances that have been saturated with inorganic cations, the interlayer area can be measured conveniently by liquid-state sorption methods; polyhydric alcohols such as glycerol (*3, 4*) and ethylene glycol (*5*) have been used. With the glycerol method, a saturated sample of clay is treated with an aqueous solution of glycerol and, in a glycerol-vapor atmosphere maintained at slightly less than saturation, the water and excess glycerol are removed from the clay at a temperature of 110° C. A monomolecular layer of glycerol is retained by the clay, both on the areas between the unit layers and on the exteriors of the particles. Completion of the desorption of excess glycerol is signaled by the glycerol-treated clay's attainment of constant weight. The weight increase over the oven-dry weight of the clay represents the weight of the monolayer. By use of the monolayer weight and the coverage area assigned to the glycerol molecule, the surface area of the clay is estimated.

X-ray diffraction results included in part I of this article indicated that glycerol molecules, applied in the liquid state, also can penetrate interlayer spaces when montmorillonite is saturated with any one of a number of short-chain aliphatic amine cations, containing up to four carbon atoms. The entry of glycerol was revealed by an expansion of the basal spacing of the amine-cation saturated clay, which provided a separation of not less than about 4 A. between the unit layers. Analogous restricted lattice expansion caused by sorption of certain polar molecules on amine-treated montmorillonite also has been observed (*1, 6*).

In this study, the extent of the interpillar area of a montmorillonite saturated with a series of short-chain aliphatic amine cations was estimated by means of the glycerol retention technique. The interpillar area, the total interlayer area of the untreated clay, and the number of amine cations present in each amine-saturated product, as determined by analysis, were used to estimate the area covered by the individual cations.

Summary and Conclusions

Wyoming bentonite was treated with solutions of the hydrochlorides of a number of short-chain aliphatic amines and two quaternary ammonium salts and washed with an acetone-water solution. Carbon analyses indicated that in most cases the cations were adsorbed and retained against washing in amounts consistent with the exchange capacity of the clay. However, for the one- and two-carbon amines, the adsorption was greatly in excess of the exchange capacity, although for the larger tertiary amines it was less than the exchange capacity.

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[1] Presented at the 10th National Conference on Clays and Clay Minerals, Austin, Texas, October 1961.

[2] The references indicated by italic numbers in parentheses are listed on pages 62-63.

Glycerol retention determinations indicated that the surface available to glycerol is greatly reduced by treatment with amine cations, the reduction being most pronounced for the larger cations. After allowing for glycerol retained on external surfaces, the remaining glycerol was interpreted as having been retained between the cation pillars in the interlayer spaces. From the glycerol retention figures, estimates were made of interpillar area ranging from more than 100 $m.^2/g.$ for the methylamine-treated clay to essentially zero for the clay treated with tri-n-butylamine.

Consequently, the difference in glycerol-retaining capacity between untreated bentonite and the amine-treated clay was ascribed to the interlayer area physically occupied by the amine cations. From the loss of glycerol-retaining capacity, the total area occupied by the cations and the average cross-section area per cation were calculated for each of the several amines. These experimental cross-section areas were shown to be in good agreement with coverage areas obtained from atom models of the individual amines. This agreement confirmed that the interlayer space not occupied by amine cations may be occupied almost completely by glycerol and, consequently, that the glycerol retention method provides an acceptable measure of the area between the amine cation pillars. The completeness with which glycerol fills the interpillar areas suggests that at least the larger cations possess some freedom of lateral adjustment in the interlayer space.

The distribution of cations within the monolayers was considered to be somewhat irregular, with the actual distances between adjacent cation sites deviating considerably from the calculated average distance of 7.7 A. Because of this irregularity, saturation treatments with cations such as tri-n-propylamine, which are large enough to occupy all of the interlayer surface of the clay on the basis of a complete exchange of cations, do not necessarily accomplish this coverage. On the one hand, the exchange of large cations was not complete because some of the sites that happened to be closely spaced may have been blocked; but at the same time, in regions of widely-separated sites, some of the interlayer area may still have been unoccupied. Upon glyceration a few glycerol molecules may therefore be adsorbed into the interlayer space; the probability of this presumably decreases with increasing layer charge of the montmorillonite. Because of this, saturation with triethylamine cations, which had previously been suggested as a means of preventing the penetration of glycerol into the interlayer spaces, was not completely effective with Wyoming bentonite. Estimates of the external surface of some soils and clays obtained by glycerol retention after such treatment thus may be slightly high.

Methylamine, ethylamine, and dimethylamine seemed to be adsorbed and retained by the clay in a manner somewhat different from the other amine cations, apparently being in perpendicular rather than parallel orientation with respect to the planar surface of the clay. The amount of methylamine adsorbed and retained corresponds to the number of oxygen ring cavities calculated to be present on the clay surface. It is thought that a full exchange complement of methylamine cations was partially embedded in those cavities which are associated with cation sites, and that amine in excess of this was similarly embedded in the remaining ones. The adsorption of excess amine was not accompanied by either the adsorption of chloride ions or by an observable drop in pH. Consequently, although it is clear that the excess amine was not adsorbed as molecules of amine salt, it is not certain whether adsorption was in the form of amine cations or as free amine molecules. The adsorption of the two-carbon amines was somewhat less than for methylamine, corresponding to occupation of all of the cavities associated with cation sites, plus only about half of the remaining ones. This lesser adsorption is thought to have been caused by the physical difficulty of the two-carbon amines in becoming suitably oriented for embedding in the cavities.

Preparation of Amine-Saturated Clay

The clay used in this study was Volclay brand Wyoming bentonite ground to pass the 200 mesh sieve; this clay contained about 85 percent montmorillonite and the remaining 15 percent was largely quartz and feldspar. This material was used as supplied, without fractionation or other purification. The group of amines used included a number of short-chain primary, secondary, and tertiary aliphatic amine cations and two small quaternary ammonium cations; these are listed in table 1. The clay was saturated with amine cations by using 1-normal solutions of the amine hydrochlorides prepared by neutralizing Eastman organic reagent amines with hydrochloric acid to a final pH of about 7. For saturation with the quaternary ammonium cations, 1-normal solutions of tetramethylammonium bromide and tetraethylammonium chloride, prepared from the corresponding Eastman salts, were used.

For each cation saturation, 50 milliliters of the amine salt solution (5 times the exchange capacity) was added to a 10-gram portion of the clay, and the mixture was stirred until the clay was thoroughly wetted. About 100 milliliters of distilled water was then added and the mixture was stirred vigorously for 10 minutes with a vibration mixer. The strongly-flocculated clay was sedimented by centrifugation, and the supernatant liquid was decanted and discarded. This treatment was performed five times, after which the clay was washed five times by a similar process with 150-milliliter portions of 9:1 acetone-water solution. The clay remained flocculated throughout the washing process and was easily separated from the wash liquid. The last supernatant liquid was tested with a silver nitrate solution to assure its being free of chloride ion. The washed clay was dried at room temperature, lightly powdered, and stored.

Determination of Amine Content

The several amine-saturated clay materials were analyzed for carbon content by the conventional dry combustion method. The amine content in milliequivalents per gram of clay (meq./g.) was calculated by use of the carbon content data and the theoretical carbon content of the individual amines; these data are given in table 1. For comparison with these data, the cation exchange capacity of the raw clay (0.91 meq./g.) was determined by the versenate titration method described by Jackson (*7*). The amine content of the saturated clay material varied greatly; the results fell into three groups, depending on the size of the amine used for the saturation: (1) The amine content of the clay saturated with the 1- and 2-carbon amines greatly exceeded cation exchange capacity of the clay, the content of methylamine being almost three times as great; (2) the amine content of clay saturated with amines containing from 3 to 8 carbon atoms was approximately equal to the cation exchange capacity; but (3) the amine content of clays saturated with amines containing 9 or more carbon atoms was somewhat less than the cation exchange capacity. The anomalously large amine content of the clay preparations saturated with the smaller amines will be discussed later in this article. The deficiency in amine content noted for clay saturated with the larger amines agrees with findings for other relatively large amines made by Hendricks (*8*), Slabaugh and Kupka (*9*), and others; apparently this deficiency was caused by a steric interference by the large cations—those involved early in the exchange process blocking the access of some of the other cations to the remaining exchange sites. Even after repeated treatments of the clay, some of the original cations apparently remain; therefore, the amine cations were present on the clay in amounts that are less than the full exchange capacity of the clay.

INTERPILLAR AREA

Glycerol Retention Determinations

Duplicate glycerol retention determinations were made for the untreated clay and for each of the amine-saturated and quaternary ammonium-saturated clay products, by use of essentially the same procedure reported earlier (*4*). Briefly, a 1-gram sample of clay was treated with 10 milliliters of a 5-percent aqueous solution of glycerol, the clay was brought to constant weight (the glycerol monolayer condition) by removal of water and excess glycerol, at a temperature of 110° C., in an atmosphere slightly less than saturated with glycerol vapor. The difference in the initial and final weight of the clay represented the glycerol retention or weight of the glycerol monolayer. These glycerol retention determinations are given in table 1. To provide a uniform basis of comparison, the data in terms of milligrams of glycerol retained per gram of amine-saturated clay were converted to the basis of milligrams per gram of untreated clay by using the carbon content data and the stoichiometric compositions of

Table 1.—Characterization of amine-saturated clay

Saturating cation	Carbon content, [1] based on oven-dry weight of amine-saturated clay	Amine content, based on oven-dry weight of untreated clay	Glycerol retention			Calculated interpillar surface area
			Based on oven-dry weight of amine-saturated clay [2]	Based on oven-dry weight of untreated clay	Reduction from amount of untreated clay	
	Percent	*meq./g.*	*mg./g.*	*mg./g.*	*Percent*	*m.²/g.*
Untreated clay	----	[3]	----	[4] 192	----	[5] 624
Methylamine	2.46	2.35	128.5	137.4	29	431
Ethylamine	3.22	1.46	101.5	109.2	44	331
Dimethylamine	2.97	1.34	106.4	126.8	34	394
Diethylamine	4.16	.94	70.6	75.3	61	212
Di-n-propylamine	5.56	.89	54.9	59.5	69	156
Trimethylamine	3.08	.92	103.7	109.2	43	331
Triethylamine	4.16	.93	22.5	24.6	87	33
Tri-n-propylamine	7.30	.75	26.5	29.3	85	49
Tri-n-butylamine	8.83	.70	13.9	15.7	92	1
Tri-n-amylamine	9.98	.64	[6]	[6]	[6]	[6]
Tetramethylamine	4.26	.95	85.8	91.7	57	270
Tetraethylamine	7.60	.89	22.4	24.9	87	34

[1] Average of duplicate determinations, corrected for carbon content of untreated clay. Analyses performed by L. B. Hayes, chemist, Bureau of Public Roads.
[2] Average of duplicate determinations which agreed within 1.5 mg./g.
[3] The cation exchange capacity of the untreated clay is 0.91 meq./g.
[4] Average of six determinations on a sample washed five times with a 9:1 acetone-water solution.
[5] Complete interlayer surface area of untreated clay.
[6] Not determined; explanation given in text.

the amines. All the resulting data show that the amount of glycerol retained by the clay product had been substantially reduced by the amine treatment. The reductions ranged from about 30 percent to more than 90 percent of the amount retained by the untreated clay; these reductions increased generally with the increasing number of substituent alkyl groups and the length of the alkyl chains.

An effort was made to investigate the effect of even larger cations on clay; but the attempt to obtain similar glycerol retention data for the two, next larger cations in the tertiary series, tri-n-amylamine and tri-n-hexylamine, was unsuccessful. After the saturation and washing, the samples could not be brought to constant weight by oven-drying; they continued to lose weight slowly for several weeks. This weight loss was accompanied by the evolution of a characteristic amine odor, apparently caused by the desorption of amine molecules that had been adsorbed during the saturation treatment and not removed by the washing. Because of this difficulty, no further attempts were made to characterize tri-n-amylamine- and tri-n-hexylamine-treated montmorillonite by the glycerol retention technique.

External surface

As indicated, part of the retained glycerol was considered to be present in channels between the amine cations in the interlayer spaces and the remainder of the glycerol was considered to be a monolayer on the external particle surfaces of the clay. Therefore, in estimating the extent of the interpillar area from these determinations, it was necessary to correct the total retention for the amount retained on external surfaces. In estimating the external glycerol, it seemed reasonable to postulate that the external surface areas of the several amine-saturated products were not significantly different from that of the untreated clay or from each other. The raw clay was treated directly with the amine salt solution without prior dispersion, and flocculation was maintained throughout the saturation and washing treatments. Although mechanical agitation was a necessary part of the process, disruption of the primary particles was considered insufficient to have significantly affected the amount of the external surfaces. The external surface area of the untreated Wyoming bentonite material was 27 square meters per gram (m.²/g.) determined by the conventional B.E.T. method using nitrogen sorption.[3] An external surface area of 27 m.²/g. for untreated and unfractionated Wyoming bentonite, of the same grade and from the same source as that used in this study, also was cited by Mooney, Keenan, and Wood (*10*). Therefore, regardless of cation saturation, the figure of 27 m.²/g. was used as the external surface area of the clay in all subsequent calculations made during this study.

Coverage area per glycerol molecule

When calculating surface areas from glycerol retention data it was also necessary, in effect, to assign an area of surface coverage to the individual glycerol molecules. The assignment of such a coverage area for a sorbate molecule is commonly considered to be the least certain step in calculating surface area from adsorption data, whether by the B.E.T. or liquid-state sorption methods. Previously (*4*), a coverage area of 27 A.² had been assigned to the glycerol molecule; liquid density had been assumed for the glycerol monolayer and a molecular thickness of 4.5 A., as derived by x-ray diffraction, had been employed. A basal spacing leading to this same thickness was subsequently obtained for a monolayer glycerol complex of montmorillonite (*11*). A lesser monolayer thickness, approximately 4.1 A., has been observed (*12*) for glycerol-treated specimens of montmorillonite each of which had been saturated with one of a series of small amine cations. This thickness is the same as the average thickness of the glycerol molecule in a two-layer glycerol complex and was used by Jackson (*13*) when he computed a factor for estimating surface areas from retention results for both monolayer and duolayer glycerol complexes. With Jackson's computed factor, a slightly larger coverage area of 29 A.² was calculated for the glycerol molecule.

A scale model of the glycerol molecule was assembled from a Fisher-Taylor-Hirschfelder atom model kit. Measurements of clearance thickness and coverage area were made for various conformations of the molecule, the axis of the carbon chain being maintained in an orientation approximately parallel to the assumed basal clay surface. The molecular coverage areas were measured by placing the model on a sheet of paper, drawing a smoothed outline, and estimating the area of the described figure by cutting and weighing. A number of conformations having clearance thicknesses close to the 4.1 A. and 4.5 A., obtained experimentally, were examined. It was noted that more than one conformation could lead to the same thickness, and that each conformation could have a coverage area differing somewhat from others but with the same thickness. Coverage areas determined with the model generally were in the range of 27 to 31 A.², which indicated the reasonableness of the assigned areas of 27 and 29 A.²

Inasmuch as a unique conformation could not be assigned to a given layer thickness, it was impossible to select a unique measurement of coverage area corresponding to each of the two thicknesses. Therefore, a figure of 27 A.² as the coverage area for a glycerol molecule was employed in this study for surface area computations irrespective of the monolayer thickness. Correspondingly, it was calculated that 1 milligram (mg.) of glycerol in the form of a monolayer occupies 1.76 square meters (m.²) of external surface, and 3.53 m.² of surface area when interleaved between adjacent montmorillonite layers. Similarly, 1 m.² of external surface retains 0.568 mg. of glycerol, and 1 m.² of internal surface retains 0.284 mg. of glycerol.

Glycerol Retention Results in Terms of Interpillar Area

The interpillar areas were determined by use of the glycerol retention results (shown in column 5 of table 1) and the surface coverage factors set forth in the preceding paragraphs. To determine these areas, 15.3 mg. of glycerol per gram of clay, corresponding to the external surface (27 m.²/g. × 0.568 mg./m.² = 15.3 mg./g.), was subtracted from the glycerol determinations and the remainder multiplied by the factor for glycerol on internal surfaces, 3.53 mg. The results for both untreated clay and clay treated with each of the saturating cations are shown in the last column of table 1. Because of the natural impurities in the bentonite, the area of 624 m.²/g. is considered reasonable for the interlayer surface area of the untreated clay, although it is less than the area of from 760 to 800 m.²/g. commonly

[3] This determination was kindly performed by Dr. Paul Seligmann, Portland Cement Association.

ascribed to pore monolayers. All these data indicate that the interlayer area available to glycerol after the retention of amine cation pillars is substantially less than that of the untreated clay. These areas are approximately 400 m.² g. for methylamine and dimethylamine saturated clay; 300 m.² g. for saturations with ethylamine, trimethylamine, and tetramethylamine; 150 to 200 m. g. for diethylamine and di-n-propylamine; and less than 50 m. g. for the larger tertiary amines and tetraethylamine. For clay saturated with tri-n-butylamine, it is apparent that no remaining interpillar surface is available to glycerol, the retention of 15.7 mg. g. being essentially the same as the 15.3 mg. g. expected for the external surface of the clay.

Some interlayer space apparently still was available for glycerol sorption even when the montmorillonite had been treated with such relatively large cations as triethylamine and tetraethylamine, for which an essentially complete exchange of cations had been accomplished; and with tri-n-propylamine, for which the exchange had been somewhat less than the full exchange capacity. But, with the tri-n-butylamine, the more limited amine cation population apparently had occupied the interlayer space to such an extent that glycerol molecules were unable to enter.

Interpillar Coverage

An explanation of the interpillar coverage must rest on a realistic consideration of the distribution of cation sites with respect to the planar surface of the clay. Each cation in the monolayer of the amine-saturated clay is bonded to one of the two adjacent clay surfaces and occupies an area equal to its own cross-section on the surface to which it is bonded and at the same time it occupies an equal area on the other clay surface with which it is in contact. From the total surface area of the original clay and the cation exchange capacity (table 1), the average area available per exchange site was calculated as 120 A.²; consequently, half this area, 60 A.², is the cross-section area available per site. Based on two-thirds of a charge per unit cell, the theoretical total and cross-section area available for exchange per unit cell of montmorillonite would be 140 A.² and 70 A.², respectively. Therefore, as a first approximation, the cation positions may be visualized as distributed over the planar surface in the form of a rectangular net with a unit mesh area of 60 A.² in which the nodes, representing the centers of the cation postions, are 7.7 A. apart.

Obviously, for a glycerol molecule to enter an incompletely packed monolayer, such as that described in the preceding paragraph, sufficient room must be available between the individual cations. Examination of an atom model indicates that the smallest width of the glycerol molecule in an interlayer complex is approximately 5 A. Accordingly, if the cations were fixed in rigid positions 7.7 A. apart, a glycerol molecule could enter between them only if the width of the cations was less than 2.7 A. As this critical width is much smaller than the Van der Waals diameter of a methyl group, saturation of the clay with any of the amine cations used in this study should almost exclude glycerol molecules from the interlayer spaces. But data from this study indicated that with most of the cations used for saturation, substantial numbers of glycerol molecules did enter and were retained in the interlayer spaces of the saturated clay. Thus, considering the distribution of cation positions in the monolayer as being of a fixed, geometrically-regular pattern is clearly inadequate.

If the individual cations were considered to have a degree of mobility sufficient to permit the entrance of some glycerol molecules, the process generally would be more in accord with the experimental results. However, unless the sum of the diameters of the cation and the glycerol molecule happened to be fortuitously close to the distance between cation sites, the resulting, mixed monolayer would still have a considerable unoccupied space, as the lateral adjustment of a cation to widen a channel sufficiently to accommodate a glycerol molecule would correspondingly constrict the opposite channel.

Possibly a more realistic consideration of the cation distribution is that the interlayer cations are located in association with the cavities of the oxygen rings of the silica sheets. Although the sources of charge deficiency in the octahedral or tetrahedral layers are not centered directly below these cavities, it has been demonstrated that small cations tend to enter the cavities when the clay is dried. There are two cavities for each planar face of the montmorillonite unit cell, and the total number for a montmorillonite of a given surface area may readily be calculated. The cation exchange capacity of the experimental montmorillonite suggests that slightly more than one-third of these cavities may have cations associated with them. It may be assumed that, although the arrangement of cavities themselves is highly regular in the silica sheets, the distribution of those that are associated with cations should be completely random. Moreover, because in a packet of unit layers, the stacking with respect to the *a* and *b* axes is known to be essentially random, the upper and lower oxygen rings that face on a given interlayer space are not normally superposed. Because of these factors, some of the cation sites must be quite closely spaced and others widely separated.

Accordingly, for saturation with an amine cation of moderate size, those cations occupying the more closely spaced sites might themselves effectively fill the space between the sites, although those on the more widely spaced sites might leave empty channels of much greater width than would be expected on the basis of a uniform spacing of the cations. A continuous distribution of channel widths would thus be expected, ranging from zero to relatively large dimensions. If the cations were fixed in position, some channels would be too narrow to accommodate glycerol molecules; however, if a certain degree of mobility of the cations is permitted to allow for lateral readjustment during sorption of the glycerol, the resulting mixed, glycerol-amine cation monolayer might be almost completely efficient in occupying the interlayer space. On this basis, the area occupied by the pillars may be calculated as the total interlayer area (65 m.²/g. for clay of this study) minus the area found to be covered by glycerol molecules. Dividing this estimate of pillar area by the number of pillars present would yield a figure for the area occupied by an individual pillar and half of this would represent the cross-section area of the pillar. Any vacant area remaining in the mixed monolayer would, on this basis, be erroneously allocated to cation occupation and, if the amount were significant, the cross-section areas determined experimentally in this study would be too high. Comparison of such experimental data with data obtained from atom models should provide a test of whether the vacant channel area is in fact negligible, and thus whether the glycerol retention technique provides an accurate measurement of interpillar surface area.

Calculation

In order to make this calculation for a given amine-treated clay it is necessary to estimate the number of cation pillars that are present. The total number of amine cations is readily calculated from the amine content, but it is necessary to correct for the cations that are present on external surfaces. Grim (*14*), apparently citing Hendricks, Nelson, and Alexander (*15*), suggested that about 80 percent of the exchange capacity of montmorillonite is associated with planar surfaces, the remainder being due to broken bond sites on the edges of the crystallites. However, A. Weiss (*16*) more recently obtained experimental evidence establishing that, for mica, vermiculite, and montmorillonite, cation exchange at near neutral pH values is associated exclusively with planar surfaces. If Weiss' conclusion is adopted, the external portion of the cation exchange capacity may then be estimated from the internal and external surface areas by assuming (1) that the distribution of the cation exchange sites is the same for all planar surfaces, whether internal or external, and (2) that as clay crystals are usually much larger in the *a* and *b* directions than in the *c* direction, most of their external surface is of the planar type, and the small proportion of exchange sites attributed to the edges of the crystals may be ignored. As has been indicated, the external surface of the clay used in this study is 27 m.²/g., which is about 4 percent of the total surface. According to these assumptions, the external amine content would also bear this same relationship to the total amine content and, consequently, the internal or interlayer amine content would be about 96 percent of the total amount of amine found.

The interlayer area considered to be occupied by the amine cations was calculated for each of the several amine-treated products on the assumptions given in the preceding paragraph, and the determinations are listed in the second column of table 2. Also listed in table 2 are figures for the total number of cations per gram of clay, the estimated number

of pillars (96 percent of the total), the amount of interlayer area occupied per pillar, and the cross-section area of the pillar. The experimental estimates of cross-section area ranged from 8 A.^{2} for methylamine to 77 A.^{2} for tri-n-butylamine and, as expected, increased more or less regularly with increasing chain length and number of substituent alkyl groups.

Comparison of Estimates

To provide for the proposed comparison of the estimates of interlayer area with corresponding cross-section areas derived from atom models of the cations, a set of the models was assembled from the Fisher-Taylor-Hirschfelder kit. These models are designed to reproduce accurately the essential geometric features of the cations on a scale of 1 centimeter (cm.) to 1 A. Outline tracings of the coverage areas were made on paper, as illustrated for tri-n-propylamine in figure 1. As originally traced, the outlines contained a number of bulges and indentations, as indicated by the dashed line, but it was thought that smoothing and relaxing the outline, as indicated by the solid line, would provide a more realistic measurement. Areas enclosed within the outlines drawn on this basis were estimated by cutting and weighing, and are listed in the last column of table 2. Two types of orientation were used in positioning the models for outlining the coverage areas. For most of the cations, the orientation giving the minimum height was used; that is, with the carbon chain parallel to the planar surface of the clay. However, for the three smallest cations, which were thought to be partially embedded in the oxygen ring cavities, the areas were outlined when the models were oriented with the long axis perpendicular to the planar surface. Outlined areas for the three cations in parallel orientations were substantially higher than those cited in the table, 16 A.^{2} for methylamine and 24 A.^{2} for dimethylamine and ethylamine. Obviously, for these three amines the experimental estimates of cross-section area were in accord with those outlined with a perpendicular rather than a parallel orientation. The matter of perpendicular orientation for these three cations is discussed in greater detail later in this article.

In general, there was a very good agreement between the cation areas derived experimentally and those derived from the models, the maximum deviation being only about 15 percent, except for methylamine. There was no indication that the experimental areas were systematically greater than those from the models, as would be expected if a substantial amount of vacant area were mistakenly ascribed to amine cation coverage because of incomplete penetration by glycerol. This consistently good agreement suggests that, for saturation with each of the amine cations, the amine cations and glycerol molecules occupied substantially all of the interlayer space, and it substantiates the validity of the measurement of interpillar area by glycerol retention. The virtually complete filling of the interpillar space by glycerol also supports the inference that, in general, the amine cations must have a considerable degree of mobility about the exchange sites; otherwise, they could hardly have arrived at such an efficient packing with the glycerol molecules.

Effectiveness of Triethylamine

In a previous study of the glycerol retention of soils and clays, Kinter and Diamond (*17*) used triethylamine saturation as a pretreatment designed to prevent the entry of glycerol into the interlayer spaces. From this study, however, although a full exchange complement of triethylamine cations was shown to have been adsorbed, the entry of a small amount of glycerol nevertheless appeared to have taken place. The glycerol retained after triethylamine saturation thus was not a strict measure of the external surface of this montmorillonite. In clays of a higher charge density, because of their larger number of exchange sites per unit of surface area, such widely-separated sites are less likely to occur than in the Wyoming bentonite. Therefore, a cation such as triethylamine might be large enough to fill completely the interlayer spaces of a clay with a higher charge density. As, in general, only a very small part of the internal surface of any montmorillonite is likely to be mistakenly classed as external surface by this procedure, the estimation of the amount of internal surface present in a given sample and, consequently, the content of expanding minerals should not be seriously in error.

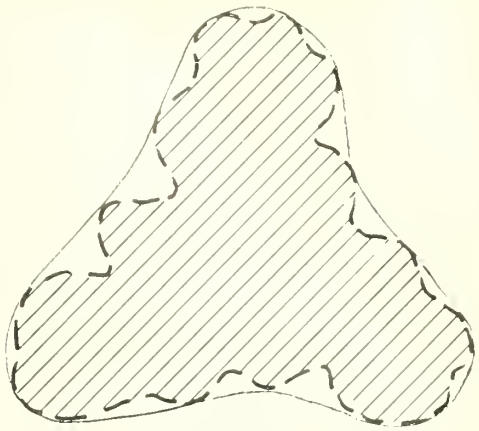

Figure 1.—Characteristic outline of tri-n-propylamine cation model is represented by dashed line. The relaxed, solid outline was used to measure the cross-section area of the cation.

Amine Content in Excess of Exchange Capacity

The carbon analysis data showed that the smallest amines were adsorbed in amounts well above the cation exchange capacity. This was further confirmed by ignition loss determinations interpreted by the method of McAtee (*18*). Morel (*19*), however, has reported the adsorption of methylamine only in amounts equal to or less than the exchange capacity, as did Cowan and White (*20*) for ethylamine. Grim, Allaway, and Cuthbert (*21*) and Cowan and White (*20*) have reported excess adsorption, but only for somewhat larger amines than the three small amines used in this study. Cowan and White (*20*) considered that the excess amine must be present either as molecules of the free amine or of the amine salt. Finding by analysis that their amine-treated clay did not contain adsorbed anions, they concluded that there had been no adsorption of amine salt molecules and that the excess therefore was present as free amine. Although they did not make pH measurements, they assumed that the adsorption of free amine had caused a lowering of the pH in their clay-amine salt solution mixture, as a consequence of the production of HCl according to the reaction:

$$RNH_3^+ \text{ saturated clay} + RNH_3Cl \rightleftarrows (RNH_3^+ \text{ saturated clay})(RNH_2) + HCl$$

Grim and coworkers (*21*) found that sorption of excess amounts of dodecylamine from a dodecylamine acetate solution was accompanied by a drop in pH from 7 to 5.6. In this study, pH measurements were made on the mixture of dry clay and 1-normal methylamine hydrochloride solution, but the initial pH of 7 remained essentially unchanged.

Table 2.—Experimental estimates of cation cross-section areas, and comparison of these areas with areas measured from outline of models

Cation	*A* Estimate of interlayer clay surface covered by cation pillars [1]	*B* Total No. of amine cations per g. of clay $\times 10^{20}$	*C* No. of cations assigned to interlayer spaces per g. of clay $\times 10^{20} = B \times 0.96$	*D* Interlayer clay surface covered per cation pillar $= A \div C$	Experimental estimate of cross-section area per cation $= D \div 2$	Coverage area per cation from atom model outline
	m.^{2}*/g.*			A.^{2}	A.^{2}	A.^{2}
Methylamine	193	13.26	12.73	15.2	8	11
Ethylamine	293	8.78	8.43	34.8	17	17
Dimethylamine	230	8.07	7.75	29.7	15	17
Diethylamine	412	5.65	5.42	76.0	38	36
Di-n-propylamine	468	5.10	4.90	95.5	48	47
Trimethylamine	303	5.52	5.30	57.2	29	29
Triethylamine	601	5.57	5.35	112.3	56	48
Tri-n-propylamine	575	4.54	4.36	131.9	66	64
Tri-n-butylamine	623	4.20	4.03	154.6	77	83
Tetramethyl-ammonium	354	5.79	5.56	63.7	32	28
Tetraethyl-ammonium	590	5.36	5.15	114.6	57	50

[1] Original interlayer surface of clay (624 m.^{2}/g.) minus interpillar surface covered by glycerol molecules.

To determine whether the amine content of the methylamine-treated clay, prepared and washed as previously described, was merely an intermediate stage in the process of removal of excess amine by hydrolysis, a newly-prepared methylamine-treated clay was subjected to successive centrifuge washing treatments with the acetone-water wash solution, and a portion of the clay was removed at each stage and analyzed by ignition for methylamine content. The results indicated that (1) 3.0 milliequivalents of amine per gram of clay were present on the treated product prior to washing; (2) after a single washing, the amine content had dropped to 2.1 milliequivalents per gram; and (2) as many as nine additional centrifuge washing treatments had not caused any further significant removal of the amine. Therefore, the amine obviously does not undergo significant hydrolysis on repeated washing; and thus, the washed sample whose carbon analysis is reported in table 1 must have been at equilibrium with respect to the amount of amine present. An x-ray spectrometric analysis of the same material indicated that it was essentially chloride-free. To check the possibility that chloride might initially have been adsorbed during the treatment and subsequently removed by the washing, the methylamine hydrochloride treatment and washing sequences were repeated on a fresh portion of the clay and the supernatant solutions analyzed for chloride. There was no indication of any adsorption of chloride by the clay at any stage.

Nature of adsorbed amine

In attempting to ascertain the nature of the excess adsorbed amine, it seemed reasonable to assume (1) that a complete exchange complement of amine cations was present in the methylamine-treated clay and (2) that amine in excess of this exchange complement could be present in one or more of three forms: additional amine cations, free amine or hydrated amine molecules, or un-dissociated molecules of the amine salt. For the excess to be present as amine cations would require that either an equal number of anions (chloride or hydroxyl) also be adsorbed to preserve the electrical neutrality of the solution or that the neutrality be preserved by a rather unlikely partial decomposition of the clay. As the analysis showed no adsorption of chloride, the excess definitely was not in the form of amine salt. Furthermore, because the pH was not lowered as would have occurred if hydroxyl ions had been adsorbed, it does not seem likely that the excess could have been in the form of cations. The remaining possibility—that the excess was free amine or hydrated amine molecules—also was not supported by the experimental data. If the excess adsorbed amine had been in the form either of free amine or of hydrated amine molecules, the reaction suggested by Cowan and White (*20*) would have applied, and a marked lowering of pH should have been observed. The specific form of the excess amine therefore remains in doubt.

Although the adsorption of large amounts of long-chain amines and quaternary ammonium compounds on montmorillonite commonly has been attributed to Van der Waals bonding, it would not be expected that the very small amines could be held by such bonding against repeated washing of the clay or drying at 110° C. It would seem, therefore, that the retention of these small amines must involve an additional mechanism. One possible mechanism, as suggested by A. Weiss (*16*) is that free ammonia or small amine molecules could bond to montmorillonite by replacing water molecules in octahedral coordination around aluminum atoms exposed at the edges of the clay crystallites. Weiss determined experimentally that approximately 0.2 to 0.3 milliequivalent of ammonia per gram of montmorillonite can be held in this way. His mechanism thus does not account for the determination in this study of approximately 1.4 milliequivalents of excess methylamine.

An explanation more in accord with the experimental data of this study is that the adsorbed amine material, consisting of both the complement of cations equivalent to the exchange capacity and the excess amine, might have been partially embedded in the oxygen ring cavities of the clay surface. It is thought that such embedding may have accounted for the ability of the clay to retain the excess amine against the subsequent washing and drying treatments. The embedding would involve orientation of the amine more or less perpendicularly to the clay surface. This embedding and perpendicular orientation is in accord with the results (*22*) obtained from oscillating-heating x-ray diffraction experiments; in these experiments the methylamine and dimethylamine cations adsorbed by montmorillonite from solutions of moderate concentration were so located and oriented. For the methylamine, this orientation presumably would have either the amine group or the methyl group partially embedded in the cavity, with the remaining group located directly above, and the long axis of the amine would thus be perpendicular to the clay surface. A similar arrangement may be visualized for ethylamine except that, as the carbon-carbon bond of the ethyl group must maintain an approximately 110° angle with the carbon-nitrogen bond, the terminal group of the ethylamine must be offset from the perpendicular axis. For dimethylamine, as the amine group is in the middle of the molecule, one of the terminal methyl groups must be embedded in the cavity, the amine group must be located directly above, and the other methyl group must be offset to maintain the required bond angle.

As previously noted, the experimental cross-section areas given in table 2 strongly support this perpendicular orientation for these three amines. Also, as indicated in part I, x-ray diffraction measurements of the separation between the clay layers in montmorillonite saturated with these three amines were consistent with perpendicular orientation and embedding. Although the long dimensions of the amines as measured from models were from 1.6 to 2.3 A. greater than the observed distances between the clay layers, the 2.4-A. depth of the cavities (*23*) is sufficient to permit this perpendicular orientation.

The specific amount of methylamine adsorbed also suggested an association of the amine with the cavities. From the widely-accepted Hofmann-Endell-Wilm structure, it may be calculated that there are approximately 1.6×10^{21} cavities per gram of pure montmorillonite. The analysis given in table 2 for the methylamine-saturated bentonite indicates that the amine content was equivalent to approximately 1.3×10^{21} methylamine cations per gram of clay. The bentonite used was not fractionated or otherwise purified; from the observed interlayer surface of 624 m.²/g. as compared to the theoretical 760 m.²/g. for pure montmorillonite, the bentonite was estimated to contain about 82 percent of montmorillonite. Accordingly, the expected number of cavities per gram of the bentonite would be about 1.3×10^{21}, which is in agreement with the amount of amine indicated to be present by the carbon analysis. It is noted, however, that for dimethylamine and ethylamine the estimates of the total number of amine cations per gram of clay, 0.81×10^{21} and 0.88×10^{21}, were considerably less than this. According to this interpretation, the cavities associated with exchange sites would have been fully occupied by amine cations, and the excess amine would have been sufficient to occupy only slightly more than half of the remaining cavities. In view of the restricted layer separations indicated by x-ray diffraction, even for wet amine-treated clay, the failure of the two-carbon amines to occupy all of the available cavities may have been due to the purely physical difficulty of becoming oriented so as to be able to enter the cavities.

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Materials and Construction AASHO Road Test Film

A 16-mm. color film on the AASHO Road Test has been released by the Bureau of Public Roads. The 27-minute film, entitled *Materials and Construction,* describes the engineering characteristics of materials, the construction methods, and the quality control measures employed in building the facilities for the Ottawa, Ill., research project.

The AASHO Road Test comprised a huge outdoor research laboratory provided to study the behavior of pavements of varied composition and thickness and bridges of varied design under the application of controlled weights and frequencies of traffic. Extensive efforts were made to construct highly uniform facilities for the Road Test in order to relate performance of the pavements directly to design and loading. The film illustrates the several unusual construction methods employed and includes numerous animated graphs and charts showing materials' characteristics and the results of quality control testing.

The film was produced by Public Roads in cooperation with the Highway Research Board, which administered and directed the project for the sponsor, the American Association of State Highway Officials. The project was financed by the States, Public Roads, and industry with the cooperation of the Department of Defense. Public Roads photographers made a complete motion picture and still photo record of the Road Test from the beginning of construction in 1956 until the end of post-traffic testing in 1961. The present film covers the 1956–1958 period.

A second film, *Pavement Research,* is in production. This film will describe the tests made on the rigid and flexible-type pavements, the rationale for analysis of data, and the principal test results. Finished prints of this second film will be available early in the fall.

Prints of the film on *Materials and Construction* are available on a loan basis from the Bureau of Public Roads, Photographic Section, 1717 H Street NW., Washington 25, D.C. These prints may be borrowed by any responsible organization. There is no charge other than for express or postage fees. Requests should be submitted well in advance of the desired showing date, and alternate dates should be indicated, if possible. Immediate return is required. Inquiries about purchase of the film should be addressed to the Public Roads Photographic Section.

Resistance of Concrete Surfaces to Scaling by De-Icing Agents

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported[1] by WILLIAM E. GRIEB, GEORGE WERNER, and DONALD O. WOOLF, Highway Research Engineers

This article presents the results of the continuation of tests made by the Bureau of Public Roads as part of its investigation of materials and procedures for protecting concrete pavements against the scaling and disintegration caused by the use of calcium chloride as a thawing agent for ice removal. Data are presented on the continuation of outdoor tests in which concrete slabs that previously had been exposed for two winters, 51 cycles of freezing and thawing, were continued for three additional winters through 154 cycles of freezing and thawing. For convenience, data reported in previous articles on the results of the first two winters of testing have been repeated in tables included in this article.

Data are also presented from an investigation made of the effect that protective coatings or admixtures might have on the control or prevention of scaling of the concrete caused by de-icing agents.

Introduction

AN ARTICLE on resistance of portland cement concrete to surface scaling by a de-icing agent *(1)*[2] appeared in a previous issue of this magazine, in which the results were reported on tests of small concrete slabs subjected to outdoor weathering. At the time of the previous article, the specimens had been exposed for two winters and subjected to 51 cycles of freezing followed by thawing by calcium chloride. Subsequently, exposure of some of the specimens was continued for three more winters; the slabs were subjected to a total of 154 cycles of freezing and thawing before the tests were discontinued. As the study continued, additional tests were made on similar specimens to study: (1) the effect of various protective coatings on the top surface of the concrete, and (2) the effect of admixtures in the concrete for the control or prevention of scaling.

[1] Presented at the 41st annual meeting of the Highway Research Board, Washington, D.C., January 1962.

[2] References indicated by italic numbers in parentheses are listed on page 73.

Conclusions

Effect of air-entraining admixtures.—All of the air-entraining admixtures used were effective to different degrees in delaying the start of significant scaling. The salts of proteinaceous materials, and some of the synthetic detergents and some of the salts of wood resin, were less effective than the other air-entraining agents.

Effect of type of base.—The scaling of concrete was less pronounced when the concrete was cast in a mold with a sand base than when cast in a mold with a watertight metal base. No significant scaling was shown by air-entrained concrete cast on a sand base if the air content was more than 3 percent. Significant scaling was found on some air-entrained concrete cast on metal bases with air contents as high as 7.4 percent. For concrete containing 7.5 percent or more of air, no significant scaling developed regardless of type of base of mold. These findings are interpreted to mean that, if concrete containing the usual specified amount of air (3 to 6 percent) is placed on a waterproof base, scaling may develop if de-icing chemicals are used.

Effect of portland cement.—Concrete prepared with a low-alkali portland cement was more resistant to scaling than concrete prepared with a high-alkali cement.

Effect of slag cement.—No appreciable difference was found regarding resistance to scaling of concrete prepared with portland cement or portland blast-furnace slag cement from the same mill.

Effect of fly ash.—Concrete prepared with 5¼ bags of portland cement and 1⅓ bags of low-carbon fly ash per cubic yard was equal in resistance to scaling to the concrete containing 6 bags of cement and no fly ash; and it was superior to concrete containing 4 bags of cement and 2 bags of fly ash. Fly ash of a high carbon content generally was inferior to low-carbon fly ash for use in concrete.

Effect of type of curing. Concrete given no moist curing other than storage in room air at 50 percent relative humidity had better resistance to scaling than concrete given moist curing for 3 to 28 days. Membrane curing gave some protection against attack by calcium chloride as long as the film remained unbroken. Other types of curing materials applied shortly after the concrete was cast were of some benefit in delaying the development of scaling.

Effect of vacuum treatment.—Concrete subjected to a vacuum treatment on the exposed surface was more resistant to scaling than similar but untreated concrete.

Effect of surface treatments.—Protective surface treatments, usually applied 14 to 28 days after the concrete was cast, in general delayed but did not prevent the development of scaling.

Effect of silicone admixture.—An aqueous silicone solution used in proper amount as an admixture for concrete was effective in preventing scaling. The silicone admixture also increased the compressive strength of the concrete and caused a marked retardation of the setting time.

Effect of latex emulsion admixture.—An emulsion of polystyrene latex used as an admixture for concrete was effective in preventing scaling but the concrete so prepared contained 10 percent or more of air.

De-icing agents.—Of the several de-icing agents tested, calcium chloride was found to be the most effective and to cause less scaling of concrete.

Preventive measures.—In addition to the materials or treatments mentioned favorably in preceding paragraphs, scaling of concrete can be prevented or reduced by the elimination of waterproof coatings or sheetings on the subgrade, by the reduction of the water-cement ratio, and by permitting the concrete to dry before it is first frozen. For concrete placed shortly before freezing is anticipated, wet curing of the concrete should be minimized.

Description of Tests

A general description of the outdoor exposure test is, as follows:

● The slabs were 16 by 24 by 4 inches in depth with a raised edge or dam around the perimeter of the top surface. Except where noted, the slabs were cast in watertight molds with metal bases. The top surface of each was given a light broomed finish about 2½ to 3

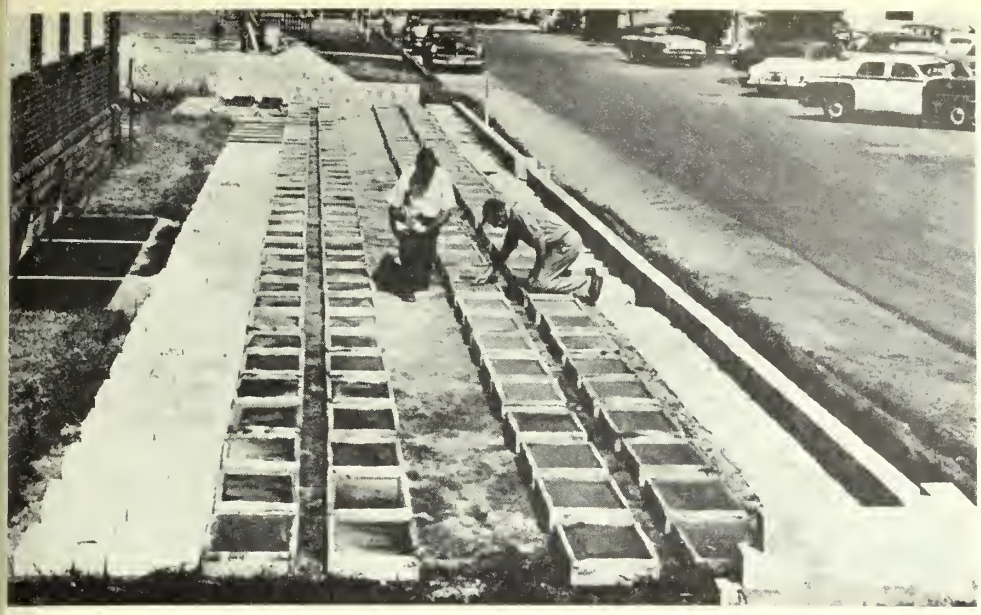

Figure 1.—Exposure area.

ours after molding and just prior to the pplication of the initial curing, if such was pplied. Except when other methods of initial uring are noted, the slabs were covered with et burlap for 24 hours and were moist cured r 2 or more days. The slabs were placed the outdoor exposure area usually at an ge of 28 or more days. Sometimes freezing nd thawing was started immediately after xposure and sometimes it was started as uch as 6 months later.

● The materials used were type I or IS ements, siliceous sands having a fineness odulus of 2.70 to 2.90, and a crushed lime-tone of 1 inch maximum size. The concrete as mixed and the slabs cast in accordance ith standard laboratory procedures. The lump was 3 inches, plus or minus one-half inch, nd, except where noted, the cement content as 6 bags per cubic yard. Both air-entrained nd non-air-entrained concretes were included n these tests. The air content for air-ntrained concrete was usually 5½ percent, lus or minus 1 percent.

● During cold weather when freezing was xpected, the top surface of each slab was overed with one-fourth to one-half of an inch f water. The next morning after the water ad frozen, commercial calcium chloride flakes ere applied uniformly at a rate of 2.4 pounds er square yard of surface. After the ice had elted, usually about 3 or 4 hours later, the urface of the slabs was flushed with water rom a hose to remove the chloride solution nd to leave fresh water for the next freezing. nly one cycle of freezing and thawing was btained each day. A view of the exposure rea is shown in figure 1.

● The slabs were examined at intervals and ated for surface scaling. The ratings were ased on visual observations of the extent and epth of the scaling. The slabs were washed horoughly and loose mortar removed from the surface prior to rating. A general de-scription of the numerical ratings is as follows:

0 no scale.

1—scattered spots of very light scale.

2—scattered spots of light scale with mortar surface above coarse aggregate removed.

3—light scale over about one-half of the surface.

4 light scale over most of the surface.

5—light scale over most of the surface, with a few moderately deep spots, where the mortar surface was below the upper surface of the coarse aggregate.

6—scattered spots of moderately deep scale.

7—moderately deep scale over one-half of the surface.

8—moderately deep scaling over entire surface.

9—scattered spots of deep scale with the mortar surface well below the upper surface of the coarse aggregate; otherwise moderately deep scaling.

10—deep scale over entire surface.

A rating of 5 or more would indicate significant or major scaling. The ratings given the slabs were based on the judgment of different observers at the various times that the observations were made, which accounts for occasional slight reversals. Typical examples of slabs with ratings of 2, 5, 7, and 10 are shown in figure 2.

RESULTS OF CONTINUATION OF PREVIOUSLY REPORTED TESTS

The outdoor exposure tests that were previously reported (*1*) through 51 cycles of freezing, followed by thawing with calcium chloride, were continued through 154 cycles. The results of these tests are given in tables 1 to 5 inclusive and continue the presentation of data reported in tables 16, 17, 19, 20, and 21 of the previous article. In addition, ratings are given here for 79, 131, and 154 cycles. At the end of each winter, many of the specimens that showed significant scaling, considered to be a rating of 5 or more, were discarded to make room for additional specimens.

Air-Entraining Admixtures

The effects of air-entraining admixtures on the resistance of concrete to scaling are shown in tables 1 and 2. All of the non-air-entrained slabs had severe scaling at 17 cycles and were given a rating of 10 (deep scaling over entire surface) by 39 cycles. With some exceptions, the slabs with the air-entraining admixtures had excellent resistance to the action of calcium chloride. Poor resistance to scaling was shown by the slabs cast on metal bases that had been prepared with the admixtures classified as salts of proteinaceous materials, some of those classified as salts of wood resin, and some classified as synthetic detergents. No tests were made to determine the relative size and number of entrained air voids caused by the various types of air-entraining admixtures. Large voids might explain the poor resistance that was shown by some of these concretes.

Table 1 gives the results for slabs cast on metal bases, and table 2 gives the results for slabs cast on sand bases. A comparison of the data given in tables 1 and 2 shows that, generally, the action of calcium chloride was less severe on air-entrained concrete cast on sand bases than on similar concrete cast

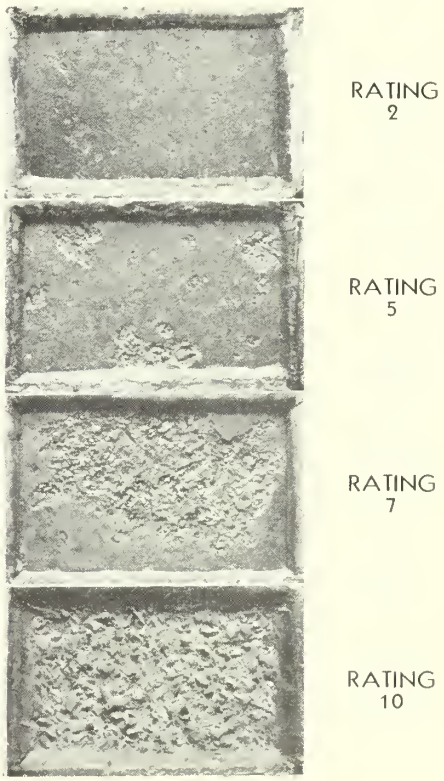

Figure 2.—Examples of typical ratings used to evaluate surface scaling of specimens subjected to outdoor freezing and thawing by calcium chloride.

on watertight metal bases. Sixteen of the 55 slabs cast on a metal base had significant scaling. Only one of the 16 slabs cast on a sand base had similar scaling, but the concrete of this slab contained only 2.6 percent of air.

Half of the specimens, as shown in tables 1 and 2, were prepared with cement A, and the other half were prepared with cement B. A comparison of the results indicates that the air-entrained slabs with cement B in general were not as severely attacked as those containing cement A. Nearly the same number of slabs prepared with each cement showed significant scaling. Of the air-entrained specimens that showed significant scaling, those prepared with cement A had air contents of 5.5 to 7.4 percent, and those prepared with cement B had air contents of 2.6 to 6.0 percent. The lower air contents for slabs with cement B tended to mask the effect of this cement on the concrete's resistance to scaling. The data indicate that, for equal air content, slabs prepared with cement B had better resistance to scaling than those prepared with cement A. One significant difference between the cements was the alkali content. Cement A had a high alkali content of 1.09 percent, expresse as sodium oxide, and cement B had an alka content of only 0.14 percent. The chemic analyses of these two cements are given table 1 of reference *1*. These slabs of ai entrained concrete had very little addition scaling after 51 cycles. Only once was t increase in numerical rating more than tv for specimens carried through 154 cycl

Use of Fly Ash

The effect of fly ash, as a replacement f part of the cement, on the resistance of co

Table 1.—Effect of air-entraining admixtures on resistance of concrete to scaling for slabs cast on metal bases [1]

Admixture number [3]	Cement A: Air, percent	Cement A: Rating after freezing and thawing [2] for cycles— 12	17	39	51	79	131	154	Cement B: Air, percent	Cement B: Rating after freezing and thawing [2] for cycles— 12	17	39	51	79	131	154
Base Mix; No Air-Entraining Admixture																
None	1.8	6	10	10	10	out	------	------	1.0	4	8	10	10	out	------	------
Do	2.1	1	5	10	10	out	------	------	2.1	8	10	10	10	out	------	------
Do	1.8	8	10	10	10	out	------	------	1.1	8	10	10	10	out	------	------
Do	1.9	6	10	10	10	out	------	------	1.1	8	10	10	10	out	------	------
Do	1.0	8	10	10	10	out	------	------	0.6	3	10	10	10	out	------	------
Salts of Wood Resin																
1	6.1	1	1	1	2	2	2	2	5.7	0	1	1	1	3	3	3
2	7.3	1	1	3	6	out	------	------	5.7	1	1	2	3	4	4	5
3	6.0	1	1	2	2	3	3	3	6.5	1	1	2	2	2	2	2
4	6.1	1	3	6	7	out	------	------	6.0	1	1	2	3	4	5	5
6	4.9	1	1	1	2	3	3	3	6.6	1	1	2	2	3	3	3
7	6.3	1	1	1	1	3	3	4	5.1	1	1	2	3	3	3	3
30	5.7	1	1	1	2	2	2	2	7.8	2	2	2	2	3	3	3
31	4.0	1	2	2	3	4	4	4	5.6	1	2	2	3	4	3	4
32	6.4	0	1	1	1	2	2	2	6.4	1	1	1	2	3	3	3
Synthetic Detergents																
12	6.7	1	2	2	4	4	4	5	5.6	0	1	1	2	2	3	3
14	5.4	1	1	2	3	3	4	4	4.6	1	2	2	2	2	3	3
17	6.3	0	1	3	5	out	------	------	4.8	1	5	7	8	7	7	7
26	8.4	0	1	1	1	2	2	2	4.9	1	1	1	1	2	2	2
26	5.6	1	1	7	9	out	------	------	------	------	------	------	------	------	------	------
Salts of Sulfonated Lignin																
18	7.0	1	1	1	2	2	2	2	5.5	1	1	1	1	2	2	2
19	3.7	1	1	1	2	3	3	3	2.8	2	4	5	7	7	out	------
21	5.1	1	1	1	2	2	3	2	5.5	0	1	2	2	2	3	2
Salts of Petroleum Acids																
11	7.4	0	1	2	4	4	6	6	6.7	1	1	1	1	3	3	2
27	6.5	1	1	1	1	2	2	2	4.1	1	1	1	3	4	5	5
27	------	------	------	------	------	------	------	------	5.1	1	1	2	2	3	3	3
Salts of Proteinaceous Materials																
24	5.5	1	1	4	9	out	------	------	4.3	1	2	4	8	out	------	------
25	6.3	0	1	3	6	out	------	------	4.1	1	1	2	6	out	------	------
Fatty and Resinous Acids and Their Salts																
8	7.6	0	1	1	3	3	3	3	8.4	1	1	1	1	2	2	2
8	------	------	------	------	------	------	------	------	4.3	1	1	1	5	6	6	7
9	6.8	1	1	1	2	2	2	2	5.6	1	1	2	2	3	2	2
10	6.7	0	1	1	2	2	2	2	5.5	1	1	1	2	2	2	2
Organic Salts of Sulfonated Hydrocarbons																
23	7.5	1	1	1	2	2	2	2	5.6	1	1	1	3	4	4	4
Miscellaneous																
33	5.7	1	1	1	2	3	3	2	4.5	1	2	2	2	2	3	3
34	5.7	1	1	1	2	3	4	3	6.9	1	1	1	1	3	3	3

[1] Continuation of tests reported in table 16 of reference *1*. Slabs were made from February to June 1952, were stored in moist air for 30 to 120 days, and all slabs were stored in exposure area in July. The slabs were 6 months or more old when freezing and thawing started.
[2] Each rating represents one slab.
[3] Numbers 1 through 27 correspond to those given in reference *4*.

Table 2.—Effect of air-entraining admixtures on resistance of air-entrained concrete to scaling for slabs cast on sand bases [1]

Admixture number [3]	Cement A								Cement B							
	Air, percent	Rating after freezing and thawing [2] for cycles—							Air, percent	Rating after freezing and thawing [2] for cycles—						
		12	17	39	51	79	131	154		12	17	39	51	79	131	154
BASE MIX: NO AIR-ENTRAINING ADMIXTURE																
None	1.1	1	8	10	10	out			0.6	4	8	10	10	out		
SALTS OF WOOD RESIN																
2	5.0	1	2	2	2	3	3	3	6.0	1	1	1	2	3	3	2
SYNTHETIC DETERGENTS																
12	3.7	1	2	2	2	3	3	3	4.5	1	1	1	2	3	3	3
15	6.7	1	2	2	2	3	3	3	4.8	1	1	1	1	3	3	3
SALTS OF SULFONATED LIGNIN																
19	3.5	1	1	2	2	3	3	2	2.6	1	1	2	3	5	5	5
SALTS OF PETROLEUM ACIDS																
27	4.3	1	2	2	3	4	4	4	4.3	1	1	1	2	3	3	2
SALTS OF PROTEINACEOUS MATERIALS																
24	4.5	1	2	2	3	4	4	4	3.7	1	1	2	2	3	3	2
FATTY AND RESINOUS ACIDS AND THEIR SALTS																
10	4.3	1	2	2	2	3	3	3	4.2	1	2	2	2	3	3	3
ORGANIC SALTS OF SULFONATED HYDROCARBONS																
23	5.3	1	2	2	3	3	3	3	6.1	1	1	1	1	3	3	3

[1] Continuation of tests reported in table 17 of reference *1*. Slabs were made in June 1952, were stored in moist air for 30 days, then were stored in exposure area. Slabs were more than 6 months old when freezing and thawing started.
[2] Each rating represents one slab.
[3] Numbers correspond to those given in reference *4*.

ete to scaling is shown by data in table 3. our different fly ashes were used in these sts; each fly ash was used as a replacement r 33⅓ percent of the cement in a 6-bag mix r both air-entrained and non-air-entrained ncrete. Fly ashes A and B had carbon ntents of 0.2 and 0.6 percent, respectively; · ash X had a carbon content of 5.0 percent, d fly ash Y had a high carbon content of .2 percent.

All of the slabs prepared with non-air-trained concrete had poor resistance to tack by calcium chloride, regardless of ether fly ash was used as a replacement for rt of the cement. All showed moderately ep scaling over the entire surface, with a ting of 8, after only 17 cycles of freezing and awing. The use of air-entrainment in-eased the resistance of the concrete to aling but the addition of fly ash to the mix d not prove beneficial. Better results were btained with the fly ashes A and B, which ad low carbon contents, than with those aving a higher carbon content.

Type of Curing

Table 4 gives the ratings of concrete slabs ured by different methods. Two rounds of abs of non-air-entrained concrete and one und of slabs of air-entrained concrete were st outdoors on different days. Considerable difference in the resistance to scaling was observed between the two rounds of non-air-entrained concrete. All of the slabs in round one showed better resistance than those of round two. This difference might have been caused partly by the daily differences in atmospheric conditions such as relative humidity, temperature, wind velocity, and sunshine. Because of the poorer resistance of round two of the non-air-entrained concrete, differences in the resistance to scaling caused by curing were more apparent. Therefore,

Table 3.—Effect of fly ash on resistance of concrete to scaling for slabs cast on metal bases [1].

Cement	Fly ash [3]	Air,[4] percent	Rating after freezing and thawing [2] for cycles—						
			12	17	39	51	79	131	154
A	None	1.0	5	8	10	10	out		
	A	1.0	6	8	10	10	out		
	B	1.0	6	8	10	10	out		
	X	1.0	7	8	10	10	out		
	Y	1.0	6	8	10	10	out		
A	None	4.0	1	2	2	3	4	6	6
	A	4.6	2	3	3	4	5	5	6
	B	5.4	3	4	4	4	5	out	
	X	4.9	4	6	7	7	8	out	
	Y	4.3	4	6	6	7	8	out	
B	None	1.0	3	8	10	10	out		
	A	1.0	4	8	9	10	out		
	B	1.0	4	8	10	10	out		
	X	1.0	6	8	10	10	out		
	Y	1.0	5	8	10	10	out		
B	None	6.8	1	2	2	2	3	4	3
	A	4.0	1	2	4	4	5	5	6
	B	3.3	5	6	7	7	out		
	X	3.9	6	8	8	8	out		
	Y	5.8	6	8	8	8	out		

[1] Continuation of tests reported in table 19 of reference *1*. Slabs were made in June 1952, were stored in moist air for 30 days, then were stored in exposure area. Slabs were more than 5 months old when freezing and thawing started.
[2] Each rating represents one slab.
[3] When fly ash was used, 33⅓ percent of the cement was replaced by an equal volume of fly ash.
[4] Air content in non-air-entrained concrete was calculated.

Table 1.—Effect of curing on resistance of concrete to scaling [1]

Type base	Initial cure [2]	Surface treatment [3]	Admixture	Rating after freezing and thawing [4] for cycles— 12	17	39	51	79	131	154
NON-AIR-ENTRAINED CONCRETE [5]—ROUND 1										
Metal	None	None	None	1	1	1	1	3	3	3
Sand	do	do	do	0	1	1	1	2	2	2
Do	Burlap	do	do	0	1	1	1	3	3	3
Do	do	Lubricating oil	do	0	1	1	1	3	3	3
Do	do	None	Lubricating oil	0	1	1	1	3	3	3
Do	Paper	do	None	0	1	1	1	3	3	3
Do	Membrane A	do	do	1	1	1	1	3	3	3
Do	Membrane B	do	do	1	1	1	1	3	3	4
Do	Lubricating oil	do	Lubricating oil	0	1	2	2	4	4	5
NON-AIR-ENTRAINED CONCRETE [5]—ROUND 2										
Metal	None	None	None	1	2	4	6	out		
Sand	do	do	do	1	2	2	3	4	7	9
Do	Burlap	do	do	1	3	3	4	7	10	out
Do	do	Lubricating oil	do	0	1	1	1	3	6	8
Do	do	None	Lubricating oil	1	1	2	2	4	4	5
Do	Paper	do	None	1	2	4	9	out		
Do	do	Lubricating oil	do	0	1	3	5	8	10	out
Do	Membrane A	None	do	0	1	1	1	8	10	out
Do	Membrane B	do	do	0	1	1	2	3	4	9
Do	Lubricating oil	do	Lubricating oil	1	1	1	5	4	3	4
Do	Lubricating oil	do	do	0	1	2	3	4	3	3
AIR-ENTRAINED CONCRETE [6]										
Metal	None	None	Vinsol resin	0	1	1	2	4	3	3
Sand	do	do	do	0	1	1	1	3	3	3
Do	Burlap	do	do	1	2	2	2	4	4	4
Do	do	do	Used crankcase oil	0	1	1	1	3	3	4
Do	Paper	do	Vinsol resin	0	1	1	1	3	3	3
Do	Membrane A	do	do	0	1	1	2	3	3	3
Do	Membrane B	do	do	0	1	2	2	3	3	4
Do	Used crankcase oil	do	Used crankcase oil	0	2	2	2	3	3	4

[1] Continuation of tests reported in table 20 of reference *1*. All slabs were made outdoors in July 1952, were removed from molds after 3 days, and then were stored in exposure area. Slabs were more than 4 months old when freezing and thawing started.
[2] Each rating represents one slab.
[3] Curing was applied 1½ hours after casting. Sheet materials or burlap were removed after 3 days.
[4] Surface protective treatment was applied after 28 days.
[5] Air content of non-air-entrained concrete was approximately 2 percent.
[6] Air content of air-entrained concrete was approximately 4½ percent.

this discussion is limited to the results obtained for round two and to the air-entrained concrete.

The tests for round two on non-air-entrained concrete without curing showed that the slabs cast on sand bases had better resistance to scaling than the slabs cast on metal bases. The slabs on which paper was used for curing had the poorest resistance. The slabs on which lubricating oil was used, both for curing and as an admixture for the concrete, were the most resistant to scaling of any of the other slabs cast for round two. These protective surface treatments delayed the scaling of the concrete only slightly; this was indicated by the large differences in the numerical ratings at 51 and 154 cycles. Membrane curing materials also delayed the scaling only slightly.

No significant scaling occurred on any of the air-entrained concrete slabs. Therefore, it was not possible to distinguish between the relative effects of any of the curing methods or air-entraining agents used. The amount of entrained air, and not the means of obtaining it, appears to be the factor governing the attainment of durability.

Vacuum Treatment

Comparisons between slabs that were finished in the usual manner and those to which the vacuum process was applied are given in table 5. Non-air-entrained concrete slabs were cast in molds with metal bases and in molds with damp sand bases. The air-entrained slabs were all cast in molds with sand bases. One-half of the slabs were finished in the usual manner and the other half were given the vacuum treatment. The slabs given the vacuum treatment were struck off in the usual manner and then the entire surface of the slab was covered with a vacuum pad. A vacuum of 18 to 25 inches of mercury was applied to the plastic concrete for about 30 minutes. All slabs were then moist cured with wet burlap for 3 days. The non-air-entrained concrete slabs cast on sand bases and vacuum treated had better resistance to scaling than those that were cast on metal bases and vacuum treated. The application of the vacuum process to the non-air-entrained concrete was beneficial in reducing the amount of scaling.

The air content of the air-entrained concretes varied from 2.6 to 10 percent. Only the slab with the lowest air content of 2 percent and placed by the conventional method had poor resistance to scaling. All of the air-entrained concrete slabs that were subjected to the vacuum process had equal or greater resistance to scaling than the corresponding slabs that had been placed by the conventional method.

Summary of Previous Tests

In general, the increases noted in scaling between the 51 cycles and the 154 cycles of freezing and thawing, at which point the tests were terminated, were not great enough to cause any material change in the previous conclusions (*1*). These continued tests indicated that 50 cycles of freezing and thawing—exposure for two average winters in a climate similar to that of Washington, D.C.—usually would be sufficient to indicate the resistance of concrete test specimens to scaling caused by the use of calcium chloride.

NEW OUTDOOR TESTS

Portland Blast-Furnace Slag Cement

A comparison was made between concrete prepared with portland cement and concrete prepared with portland blast-furnace slag cement (*2*). In these resistance to scaling tests, ten cements were used: a type I port-

Table 5.—Effect of vacuum surface treatment on resistance of plain or air-entrained concrete to scaling [1]

Air, percent	Type base	Surface treatment [3]	Rating after freezing and thawing [2] for cycles— 12	17	39	51	79	131	154
1.0	Metal	None	1	1	8	10	out		
1.0	do	do	1	4	10	10	out		
1.0	do	Vacuum	1	2	4	6	7	7	out
1.0	do	do	1	4	7	10	out		
1.0	Sand	None	1	1	9	10	out		
1.0	do	do	1	4	10	10	out		
1.0	do	Vacuum	1	1	2	4	5	8	out
1.0	do	do	0	1	2	3	4	4	
2.6	do	None	1	1	2	4	4	5	out
2.6	do	Vacuum	0	1	2	2	3	3	
3.2	do	None	1	1	2	2	3	3	
3.2	do	Vacuum	0	1	1	1	2	2	
5.2	do	None	0	1	2	2	2	2	
5.2	do	Vacuum	0	1	1	1	2	2	
6.1	do	None	1	1	2	2	3	3	
6.1	do	Vacuum	0	1	1	1	1	1	
7.0	do	None	1	1	3	3	4	3	
7.0	do	Vacuum	0	1	1	2	3	4	
10.0	None	None	0	1	1	1	3	3	
10.0	do	Vacuum	0	0	0	0	1	1	

[1] Continuation of tests reported in table 21 of reference *1*. All slabs were made outdoors in September 1952, were cured with wet burlap for 3 days, and were then removed from molds and stored in the exposure area. Slabs were more than 4 months old when freezing and thawing started.
[2] Each rating represents one slab.
[3] Vacuum was applied to top surface of plastic concrete for one-half of an hour immediately after the molding.

and and a type IS portland blast-furnace lag cement from each of five mills. Portland blast-furnace slag cement is defined as an ntimately interground mixture of portland ement clinker and granular blast-furnace lag. The same clinker was used in the nanufacture of the two types of cement from he same plant. Concrete slabs were made vith mixes containing $5^1/_2$, $6^1/_2$, and $7^1/_2$ gallons of water per bag of cement. The correspondng cement contents were approximately 6.0, 5.0 and 4.2 bags per cubic yard of concrete, espectively. The air content of this concrete aried from 4.3 to 6.1 percent. These conrete slabs were moist cured for 28 days and hen were stored in the exposure area from 2 o about 40 days before freezing occurred. They were exposed to freezing and thawing hrough two winters, for a total of 55 cycles. The ratings of the slabs after 20, 35, and 55 ycles are shown in table 6. Each rating shown is an average of the ratings for two slabs. This table also shows the equivalent lkali content of the cements and the amount of slag used in the manufacture of the slag ement.

When the water content of the concrete was $5^1/_2$ or $7^1/_2$ gallons per bag, severe scaling was bserved on all slabs by the end of 35 cycles of freezing. The concretes prepared with $6^1/_2$ gallons of water were more resistant to scaling han those with the greater water content, ont these differences in scaling were insigificant. The air content of all these conretes was more than 4.3 percent. From results of these tests, it appears that, even with air-entrained concrete, the water content of the concrete prepared with types I and IS cement is of primary importance in its resistance to scaling caused by the action of de-icing agents.

For the concretes containing $5^1/_2$ gallons of water per bag of cement, considerable differences were noted between the resistance of the concretes prepared with both types of the cements from different sources. The concretes prepared with each of the two types of cement from sources A and B (all four with low alkali content) had very good resistance to scaling—much better than the concretes containing cement from the other three sources. Concretes containing the cements from sources C and E had the poorest resistance to scaling (three of these four cements had a high alkali content). This general trend indicated that cement with a low alkali content will furnish concrete with better resistance to scaling than cement with a high alkali content. However, concretes containing the two types of cement from source D did not conform to this trend; no reason was apparent for the relatively poor resistance to scaling of the concrete prepared with the cement from source D, which had a low alkali content.

From these tests very little difference was shown in the resistance to scaling of the concretes prepared with a portland cement and those prepared with the portland blast-furnace slag cement from the same source. Averages for all 10 cements, which reflect the influence of the water-cement ratio on the severity of the concrete's scaling are shown in figure 3. From these data, it appears that, to obtain resistance to frost and de-icing agents, the water content of concrete must be held to the smallest possible amount.

Table 6.—Effect of portland blast-furnace slag cement on resistance of concrete to scaling [1]

Cement				Rating after freezing and thawing [2]								
				$5^1/_2$ gallon mix			$6^1/_2$ gallon mix			$7^1/_2$ gallon mix		
Source	Type	Slag [3]	Alkali [4]	20 cycles	35 cycles	55 cycles	20 cycles	35 cycles	55 cycles	20 cycles	35 cycles	55 cycles
		Percent	*Percent*									
A	I		0.32	0	0	1	4	7	9	7	8	10
A	IS	45	.37	0	1	1	4	7	9	4	9	9
B	I		.19	1	2	2	4	7	9	6	8	9
B	IS	45	.22	1	2	2	4	9	9	6	9	10
C	I		.32	2	3	8	4	6	6	8	9	10
C	IS	25	.61	2	6	7	6	9	9	6	9	10
D	I		.21	1	2	4	8	9	10	9	9	10
D	IS	40	.22	4	6	7	7	9	10	8	9	10
E	I		.82	2	5	8	2	8	9	8	9	10
E	IS	35	.75	4	6	7	6	9	9	9	9	10

[1] These slabs cast on metal bases were made September to November 1956, were stored in moist air for 28 days, and then were stored in exposure area. The air content of the concrete varied from 4.3 to 6.1 percent, see reference 2. The slabs were from 30 to 70 days old when freezing and thawing started.
[2] Each rating is an average of observations for two slabs.
[3] Percent of slag used in manufacturing type IS cement.
[4] Equivalent alkalies as Na_2O.

Silicone Surface Coatings

Two series of tests were made to study the effect that coatings of silicone solutions on the surface of concrete have on increasing the resistance to scaling caused by de-icing agents. In the first series, ten variables were included, as shown in table 7. The concrete had an air content of 3 percent. The slabs to which no initial curing was applied were stored in laboratory air, having a temperature varying from 70° F. to 90° F. and a relative humidity of from 30 to 60 percent. Initial curing was applied approximately 3 hours after the specimens were molded, and the surface treatment was applied from 14 to 28 days later. When three coats of surface treatment were applied, the first was applied 14 days after molding, the second at 21 days, and the third at 28 days. When only one coat was used, it was applied at 28 days.

Two different silicones were used. Silicone A was soluble in mineral spirits; silicone B was a water-soluble silicone from a different producer. The silicones were used at a rate of 1 gallon of a 2-percent solvent or aqueous solution per 100 square feet of surface. This amount was recommended by each producer. The membrane curing agent was applied at a rate of 1 gallon per 200 square feet. The specimens were stored in laboratory air for 30 days prior to being placed in the exposure area, when freezing started immediately. These slabs were exposed for five winters, and they were subjected to a total of 155 cycles of freezing and thawing by calcium chloride as the de-icing agent.

Three specimens were cast for each variable. The average ratings of the three slabs for each variable after 50, 90, 107, and 155 cycles of freezing and thawing are shown in table 7. These tests showed that the specimens given 7 days of initial moist curing usually had poorer resistance to scaling than the specimens that had no deliberate initial curing. The silicone surface treatments were sometimes of benefit in reducing the amount of scaling, one application of the treatment appeared to give better resistance to scaling than three applications.

When a silicone solution was used for curing (applied about 3 hours after molding) and no other surface treatment was applied very little scaling developed. Approximately the same results were obtained when a liquid membrane-forming curing agent was used. When three coats of silicone surface treatment were used in addition to the silicone used as a curing agent, more scaling occurred but the amount was still rated as slight. Supplementary tests have shown that the solution of silicone B would not meet the water-retention requirements for liquid membrane-forming curing materials (3). No appreciable difference was

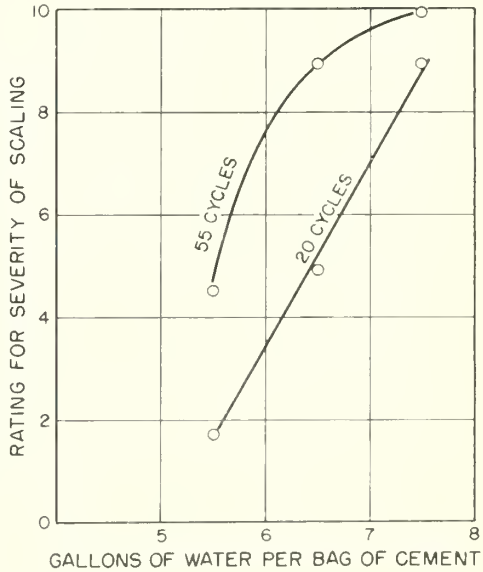

Figure 3.—Influence of water-cement ratio on severity of scaling. Averages for 10 cements.

Table 7.—Effect of silicone coatings on resistance of concrete to scaling—series one

Initial applied curing [1]	Surface treatment [3]	Rating after freezing and thawing [2] for			
		50 cycles	90 cycles	107 cycles	155 cycles
None	None	2	2	2	2
7 day moist	None	4	4	5	6
None	1 coat silicone A	1	1	1	1½
None	1 coat silicone B	1	1	1½	1½
7 day moist	do	2	2	2	3
None	3 coats, silicone B	3	2	3½	3½
7 day moist	do	3	3	4	3½
Silicone B	None	2	2	1	1½
Do	3 coats, silicone B	2	2	3	3½
Membrane	None	1	½	½	½

[1] Initial curing was applied 3 hours after molding. Slabs cast on metal bases were made in January 1955, were stored in laboratory air for 30 days, and then were stored in exposure area and immediately subjected to freezing and thawing. Air content was 3 percent.
[2] Each rating is an average of observations for three slabs.
[3] Surface treatment was applied 14 to 28 days after molding. Silicone A was mineral spirits soluble, and silicone B was water soluble.

noted between the resistance to scaling for concrete treated with silicone A and that treated with silicone B. All of the concrete slabs in this series had relatively good resistance to scaling. Only one group of slabs showed significant scaling after 155 cycles of freezing and thawing; these slabs were given 7 days of initial moist curing and no surface treatment.

A second series of tests was made to study the effect of the use of five different silicone solutions. These silicones were used as surface protective coatings, and only one application was made. They were applied 14 days after molding, at the rate recommended by the manufacturers. The five silicones included both those soluble in water and those soluble in mineral spirits, and they were obtained from three producers.

The test slabs were made on two different days, and control slabs without silicone surface treatment were made on each day. The air contents of the concretes varied from 5.2 to 6.1 percent. Half of the slabs were given no initial moist curing, and the other half were moist cured for 72 hours. The slabs were stored in laboratory air for 15 days and then were placed in the outdoor exposure area and were subjected to freezing within a few days. The slabs were exposed for 1½ winters for a total of 40 cycles of freezing and thawing.

The ratings after 10, 20, and 40 cycles are given in table 8. Each rating is an average of the ratings on two slabs for each of the silicone treatments and on three control slabs. Most of the slabs that were given no initial moist curing had better resistance to scaling than those that were moist cured for 72 hours. The use of silicone surface coating was of no benefit in preventing scaling. Except for one set of two slabs, after 40 cycles of freezing and thawing the scaling of the silicone treated slabs was greater that that of the corresponding untreated slabs. No appreciable difference was noted in the resistance of eight of the ten sets of slabs treated with the different silicones. The eight sets of treated slabs all showed significant scaling after 40 cycles of freezing, whereas none o the untreated slabs had scaling of this extent

Table 8.—Effect of silicone coatings on resistance of concrete to scaling—series two

Initial applied curing [1]	Surface treatment [3]	Rating after freezing and thawing [2] for—		
		10 cycles	20 cycles	40 cycles
None	None	½	½	1
3 days moist	None	½	1½	3½
None	Silicone C	½	1½	6
3 days moist	do	1½	3	8
None	Silicone D	1	2	6
3 days moist	do	½	2	5
None	Silicone E	0	3½	6
3 days moist	do	½	3	6½
None	None	0	½	1½
3 days moist	None	2	3	4½
None	Silicone F	½	1	2
3 days moist	do	2	3	5
None	Silicone G	1½	4	5
3 days moist	do	½	2	2½

[1] Initial curing was applied about 3 hours after molding. Slabs cast on metal bases were made November or December 1958, were stored in laboratory air for 15 days, and then were placed in exposure area and subjected to freezing within one week. Air content varied from 5.2 to 6.1 percent.
[2] Each rating is the average of two tests for the silicone treated slabs and of three tests for the control slabs.
[3] Surface treatment was applied 14 days after molding; silicones D and F were water soluble; silicones C, E, and G were mineral spirits soluble.

Although the concrete for the second series of tests had a greater air content than that of series one, its resistance to scaling was poorer. The concrete slabs of series two were subjected to freezing and thawing at an age of about 20 days whereas those of series one were subjected to freezing and thawing at an age of 30 days. The two series were made about 3 years apart and different shipments of the cement were used, which might have accounted for some of the differences in resistance to scaling of the concrete between the two series.

Silicone as an Admixture

A preliminary series of tests was made to determine the effect of a silicone solution as an admixture for concrete on its resistance to scaling. In these tests, one of the water soluble silicone solutions included in the silicone surface tests was used. The mixes used in these tests included the control mix without silicone, and mixes prepared with 0.2, 0.4, 0.6, and 0.9 percent of silicone solids by weight of the cement. The air content for the mixes varied from 4.3 to 7.0 percent

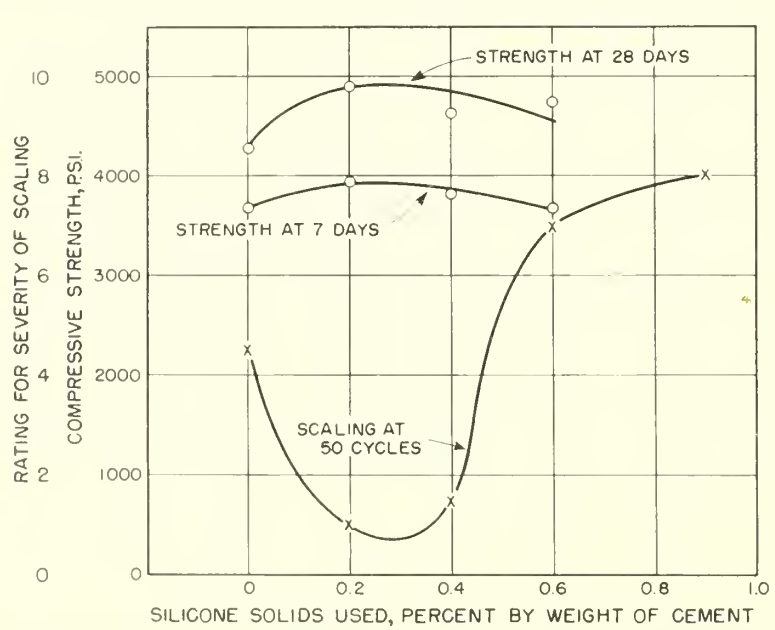

Figure 4.—Effect of a silicone admixture on scaling and compressive strength of concrete.

Table 9.—Effect of a silicone admixture o resistance of concrete to scaling

Silicone [1], percent	Air content, percent	Rating after freezing and thawing [2] for—		
		20 cycles	35 cycles	50 cycles
None	5.6	2	4	4½
0.2	5.0	½	1	1
0.4	7.0	1	1	1½
0.6	4.3	4	5	7
0.9	5.0	5	5½	8

[1] Silicone solids by weight of cement.
[2] Each rating is an average of observations for two or thr slabs. These slabs, cast on metal bases, were made in N vember 1958, were moist cured for 3 days, were stored in la oratory air for 11 days, and then were stored in exposure ar and subjected to freezing within 5 days.

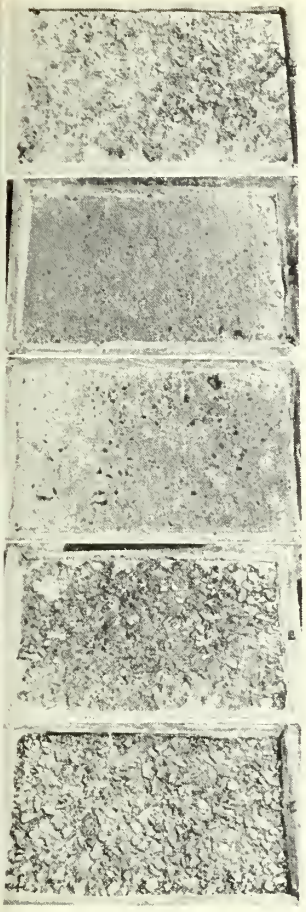
NO SILICONE ADMIXTURE, RATING 5
0.2% SILICONE ADMIXTURE, RATING 1
0.4% SILICONE ADMIXTURE, RATING 2
0.6% SILICONE ADMIXTURE, RATING 7
0.9% SILICONE ADMIXTURE, RATING 10

Figure 5.—Effect of silicone admixture on resistance of air-entrained concrete to scaling. Photographed in 1961 after 50 cycles of outdoor freezing and thawing by calcium chloride.

All of the slabs were moist cured for 3 days and were stored in laboratory air for 11 days more and then were placed in the exposure area. They were subjected to freezing within 5 days after being placed in the exposure area. These slabs were exposed for 1½ winters and were subjected to a total of 50 cycles of freezing and thawing. The ratings of these slabs after 20, 35, and 50 cycles of freezing and thawing are given in table 9.

Tests also were also made to determine the effect of the silicone as an admixture on the retardation of set and the compressive strength of concrete. The results of these tests are shown in table 10. The results of the tests for scaling and those for strength at 7 and 28 days are shown in figure 4. It will be observed that minimum scaling and maximum strength of concrete are indicated for a mix containing silicone solids in an amount of about 0.3 percent by weight of the cement. Photographs of typical slabs with silicone and the reference concrete slabs after 40 cycles of freezing and thawing are shown in figure 5.

The silicone admixture had the effect of increasing the time of set of the concrete. As shown in table 10, 0.2 percent of silicone solids required 9.0 hours for the concrete to reach initial set as measured by a Proctor penetration load of 500 p.s.i. but concrete without the silicone admixture reached this degree of set in 5.0 hours. Increasing the amount of silicone tended to increase the setting time.

Table 10.—Effect of a silicone admixture on properties of concrete [1]

Silicone [2]	Slump	Air content	Retardation of set [3]	Compressive strength at—		
				7 days	14 days	28 days
Percent	*Inches*	*Percent*	*Hours*	*P.s.i.*	*P.s.i.*	*P.s.i.*
None	2.8	5.2		3,690	3,870	4,280
0.2	3.1	5.3	4.0	3,930	4,540	4,900
0.4	3.3	5.7	6.3	3,820	4,340	4,620
0.6	2.5	4.9	5.5	3,680	4,390	4,740

[1] Proportions by dry weight were 94–205–295 pounds; cement content was 6.0 bags per cubic yard; water content was 5.8 gallons per bag.
[2] Silicone solids by weight of cement.
[3] Determined by Proctor penetration test using load of 500 p.s.i. Retardation is delay in hardening of concrete containing silicone as compared to concrete without. Concrete without silicone had initial set of 5.0 hours.

Miscellaneous Tests

A floor sealer, which had been tried experimentally by one of the State highway departments to prevent scaling, was applied as a curing agent and tested to determine its effectiveness on the resistance of concrete to scaling. The resistance to scaling of concrete cured with this material was compared with the resistance of concrete cured with wet burlap for 7 days and with the resistance of concrete cured with a liquid, membrane-forming curing material. Non-air-entrained concrete was used. The liquid curing materials were applied at two rates, 1 gallon per 100 square feet and 1 gallon per 200 square feet, approximately 2½ hours after the slabs were cast. The slabs were placed in the outdoor exposure area at an age of 14 days, and freezing did not occur for about a week. The slabs were exposed for 2 winters and were subjected to 50 cycles of freezing and thawing.

The ratings after 20, 35, and 50 cycles of freezing and thawing by the use of calcium chloride as a de-icing agent are shown in table 11. Each rating is an average of the observations for two slabs. After 35 cycles of freezing and thawing, there was no appreciable difference in the ratings of any of the slabs; none showed any significant scaling. The slabs on which the floor sealer was applied had slightly better resistance than the other slabs. At 50 cycles, all of the slabs showed significant scaling; the ones on which the floor sealer was applied still had the best resistance and the moist cured slabs the poorest. No appreciable difference was noted in the scaling between the slabs on which the floor sealer and the liquid, membrane-forming curing materials were applied at the rate of one gallon per 200 square feet and those on which they were applied at the rate of one gallon per 100 square feet.

Tests were made on the two liquids used as curing agents to determine whether they met the requirements for water retention (*3*). These tests showed that the floor sealer did not meet the requirements and that the membrane-forming compound did. The weathering tests showed slightly better results for the floor sealer than for the curing material.

Use of Emulsified Polystyrene Latex

In this phase of the investigation, emulsified polystyrene latexes were used as a curing agent, as a surface protective coating, and as an admixture for concrete. Three different latexes were used as curing agents; one was used as a protective surface treatment; and one was used as an admixture. The control specimens without latex were moist cured for 7 days. For initial curing, the latex was applied to the slabs 2½ hours after molding. Surface treatments or protective coatings were applied to the slabs at an age of 14 days. No water retention tests were made on the latexes. The concrete was non-air-entrained except when the latex was used as an admixture. Concretes prepared with the latex admixture entrained either 10 or 20 percent air without the addition of an air-entraining agent. When a latex was used as a curing material or as a surface treatment, it was applied at two rates, 1 gallon per 200 square feet and 1 gallon per 400 square feet. One slab was tested for each rate. The specimens were stored in laboratory air for 15 days then placed in the exposure area. Freezing started within a week. The slabs were exposed for 1½ winters for a total of 40 cycles of freezing and thawing.

The ratings of the slabs at 10, 25, and 40 cycles are shown in table 12. The ratings are averages for two slabs on which the same latex was applied at the two different rates. The ratings for the control slabs and for those having the latex admixture were also averages for two slabs. After only 10 cycles

Table 11.—Effect of a floor sealer and a membrane curing material on resistance of concrete to scaling

Initial curing material	Rate of application, sq. ft./gal.	Rating after freezing and thawing [1] for—		
		20 cycles	35 cycles	50 cycles
Moist [2]	-------	3	3	9½
Floor sealer	100	2	2	6½
Floor sealer	200	2	2	6
Membrane curing	100	2	3½	8
Membrane curing	200	2	3	8

[1] Each rating is an average of observations for two slabs. These slabs, which were cast on metal bases, were made in November 1956, stored in laboratory air for 14 days, and were then placed in the exposure area and subjected to freezing and thawing within a week.
[2] The slabs were moist cured for 7 days.

Table 12.—Effect of an emulsified polystyrene latex on the resistance of concrete to scaling

Initial curing [1]	Surface treatment [2]	Admixture	Air content, (calculated) percent	Rating after freezing and thawing [3] for—		
				10 cycles	25 cycles	40 cycles
Moist	None	None	2	8	9	10
Moist	Latex No. 1 [4]	None	2	4½	9	10
Latex No. 1	None	None	2	4½	5½	8½
Latex No. 2	None	None	2	3½	3½	7½
Latex No. 3	None	None	2	4	6	7½
None	None	Latex No. 1 [5]	10	0	0	1
None	None	Latex No. 1 [5]	20	0	0	½

[1] Initial curing was applied approximately 2½ hours after molding. Wet burlap was applied to moist cured slabs for 7 days.

[2] Each rating is an average of observations for two slabs. The slabs cast on metal bases were made in December 1956, were stored in the laboratory 15 days, and then were placed in the exposure area and subjected to freezing and thawing within a week.

[3] Surface treatment was applied 14 days after molding.

[4] Latex was added to the mix at a rate of 2.4 gallons per bag of cement.

[5] Latex was added to mix at a rate of 4.1 gallons per bag of cement, as recommended by the producer.

of freezing and thawing, the control slabs were severely scaled and had an average rating of 8, at 40 cycles the rating was 10.

There was no appreciable difference in the scaling of the slabs when the three latexes had been used as curing materials; the ratings after 10 cycles were from 3½ to 4, and at 40 cycles they were from 7½ to 8½. When a latex had been used as a surface protective coating, the ratings were slightly higher. These tests indicate that materials of this type, when used on the surface of the concrete, are effective only in slightly delaying the scaling. However, when a latex had been used as an admixture, excellent resistance to scaling was obtained. The excellent resistance of this concrete probably was caused by the high air content entrained by the latex. Similar results were obtained for concrete containing 10.0 percent air, as shown in table 5.

Additional Tests With Fly Ash

In the original series of tests for the resistance to scaling of concrete containing fly ash, tests were made on concrete in which one-third of the cement was replaced by an equal volume of fly ash (*2*). This is a larger amount of fly ash than usually would be used in paving concrete. In the second series of tests, the concrete was prepared with three-fourths of a bag of cement in a 6-bag mix replaced by 1⅓ bags of fly ash. The amount of sand used in the mix was reduced to compensate for the increased volume of concrete. This mix was one that had been recommended for paving concrete by a producer of fly ash.

In this series of tests, a fly ash with 0.6 percent carbon (B) and a fly ash with 11.2 percent carbon (Y) were used with two, type I portland cements. Both air-entrained and non-air-entrained concretes were used. The ratings on slabs cast from these mixes are shown in table 13; each is an average of the ratings for two slabs.

As would be expected, all of the slabs, with and without fly ash, that had an air content of only 1.0 percent showed severe scaling after only 15 cycles of freezing and thawing. Slabs containing 4.5 percent or more air showed moderate to severe scaling after 40 cycles when the high carbon fly ash had been used. The least scaling occurred on the slabs of concrete that had been prepared with cement E, fly ash B (low carbon content), and a cement content of 5¼ bags per cubic yard. The concrete prepared with this low carbon fly ash had durability equal to that of the concrete prepared without fly ash.

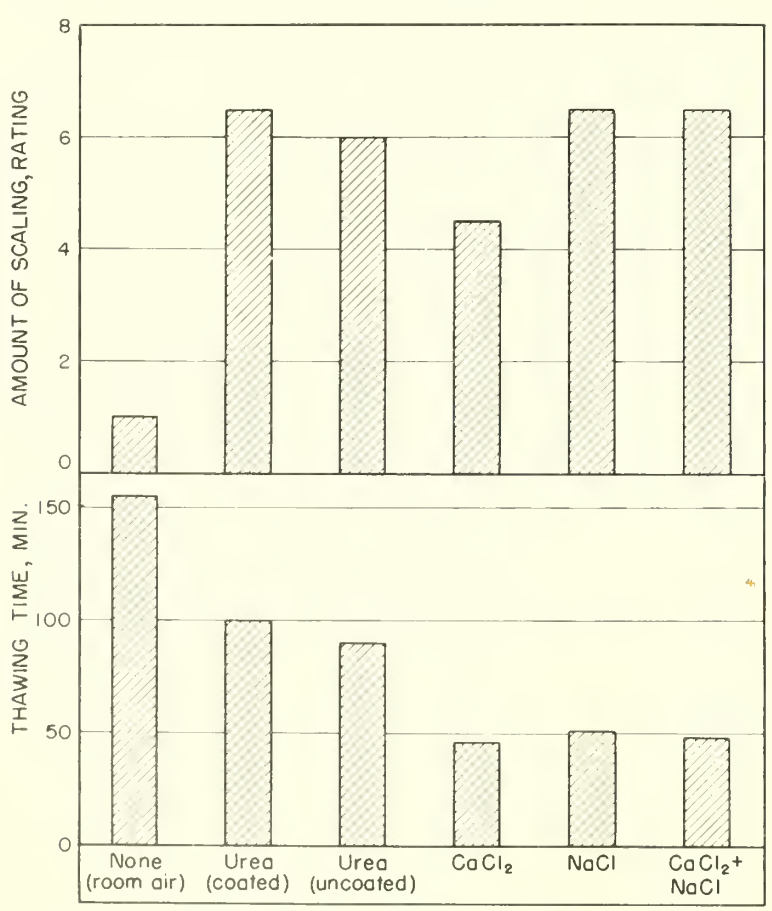

Figure 6.—Effect of de-icing agents on rate of thawing and amount of scaling of concrete containing 6 percent of air.

NEW LABORATORY TESTS

Effect of Thawing Agents

A limited laboratory series of tests was made to determine the effect of various thawing agents on the scaling of concrete and on the rate of thawing of the ice on the concrete. In this series of tests, the specimens used were similar to those used in the outdoor tests except that they were only 6 by 12 by 2 inches thick. The same mix was used for all specimens. The concrete was prepared with crushed stone coarse aggregate of one-half inch maximum size. The cement content was 6 bags per cubic yard and the air content was 6 percent. The specimens were moist cured for 7 days, and then stored in laboratory air for 14 days prior to the beginning of the freezing and thawing test. The procedure used for these tests was, as follows: A fixed volume of water was placed on the top surface of each specimen and the specimens placed in racks in the laboratory freezer; cold air at a temperature of 0° F. was circulated over the specimens. After the water had frozen, the specimens were removed from the freezer and placed in laboratory air at a temperature of 70° F. to thaw. The thawing agents were spread over the ice-covered surfaces at a rate of 2.4 pounds per square yard. After the ice had thawed, the surface was washed and fresh water was placed on the slabs. The specimens were exposed to freezing from about 4 p.m. until 9 a.m. the next day and were thawed from 9 a.m. until 4 p.m. the same day.

The length of time required for the ice to thaw and the amount of scaling on the exposed surface of the concrete were determined periodically. The results of these observations are given in figure 6. Each result given is an average for tests on five slabs. Five thawing agents were used in these tests: calcium chloride flakes, sodium chloride, a mixture of equal parts by volume of calcium chloride and

odium chloride, urea in a commercial de-icer f prilled pellets coated with clay, and urea in ncoated, shotted pellets. For comparison o thawing agent was used on one set of slabs; ney were thawed in the laboratory at room emperature. The time required to melt the e on the surface of the slabs varied from 46 inutes for the specimens on which calcium hloride had been used as a thawing agent to 55 minutes for those on which no thawing gent had been used. Calcium chloride, odium chloride, and a mixture of the two, ere much faster in thawing the ice than the gents containing either type of urea.

Of the five thawing agents, calcium chloride ppeared to be the most effective and the ast harmful to the concrete. The average ating of the specimens on which calcium hloride had been used was $4\frac{1}{2}$ after 75 cycles f freezing and thawing. Less scaling was bserved for these specimens than for the pecimens on which the other thawing agents ad been applied. The specimens on which o thawing agent had been used showed very ttle scaling and after 75 cycles they had an verage rating of 1.

Age of Concrete When Frozen

Throughout the several series of tests, the esults obtained appeared to have been influnced by the amount of water in the concrete vhen freezing occurred. The results, reported n tables 1 and 2, showed that less scaling ccurred for the specimens cast on a sand base han for those cast in a waterproof mold with metal base. There is no question but that he sand base removed some of the water equired to lubricate the concrete mix, and he test specimens so cast contained less free vater when frozen than those cast on metal ases. In other series of tests, it was not ossible to prepare all of the specimens on the ame day because of the limitations of the nolds and manpower available; consequently, ome specimens were older and probably ontained less free water than others.

Table 13.—Additional tests on the effect of fly ash on resistance of concrete to scaling

Cement			Fly Ash		Air	Rating after freezing and thawing[1] for	
Source	Type	Content	Source	Amount		15 cycles	40 cycles
		Bags/cu. yd.		*Lb./cu. yd.*	*Percent*		
B	I	6	None	None	1.0	7	10
B	I	$5\frac{1}{4}$	B	100	1.0	7	10
B	I	$5\frac{1}{4}$	Y	100	1.0	10	10
E	I	6	None	None	1.0	9	10
E	I	$5\frac{1}{4}$	B	100	1.0	9	10
E	I	$5\frac{1}{4}$	Y	100	1.0	10	10
B	I	6	None	None	5.3	$2\frac{1}{2}$	$4\frac{1}{2}$
B	I	$5\frac{1}{4}$	B	100	4.5	3	$4\frac{1}{2}$
B	I	$5\frac{1}{4}$	Y	100	4.5	$4\frac{1}{2}$	7
E	I	6	None	None	4.8	2	4
E	I	$5\frac{1}{4}$	B	100	6.2	2	3
E	I	$5\frac{1}{4}$	Y	100	5.3	5	$7\frac{1}{2}$

[1] Each rating is an average of observations for two slabs. These slabs were cast on metal bases in April 1957, were stored in moist air for 28 days, and then were stored in the exposure area. Slabs were more than 6 months old when freezing and thawing started.

No tests were made to determine the amount of the free water in the hardened concrete specimens when they were first subjected to freezing. However, the age of the concrete of all specimens at first freezing is known. If it may be assumed that the amount of free water in the hardened concrete varied inversely with the age of the concrete at first freezing, a relation of some interest may be developed. To this end, test results for all of the specimens cast on steel bases and having an air content of 4.0 percent or greater were summarized, and the percentage of significantly scaled slabs at 50 cycles of freezing and thawing were plotted against the age of the concrete at first freezing. This is shown in figure 7. Because of the marked differences in controlling variables in the various series of tests, only a general trend might be expected to be shown. This trend does indicate that concrete subjected to freezing and thawing at an early age is most liable to suffer serious scaling. From this, it may be stated that for concrete placed when freezing is imminent, scaling can be minimized if wet curing of the concrete is curtailed. The use of wind shields to permit the gradual removal of moisture from the concrete may in this case be preferable to the use of surface curing materials or wet curing methods.

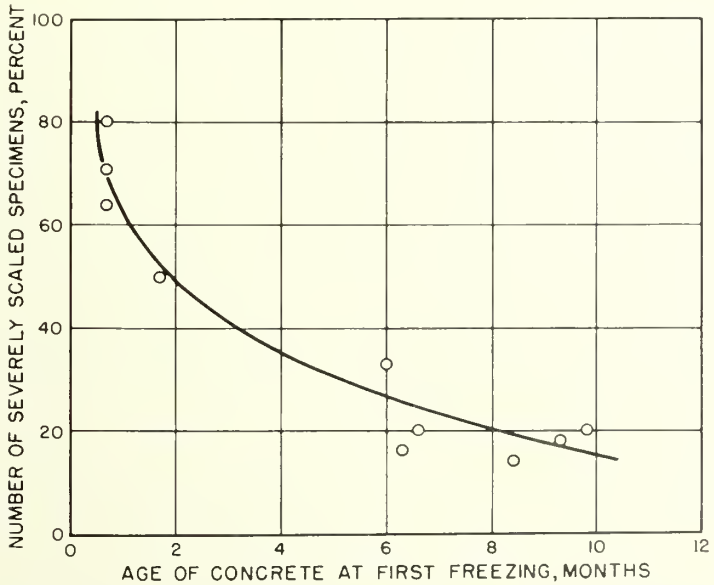

Figure 7.—Effect of the age of the concrete when first frozen on the severity of scaling.

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(1) *Factors Affecting Resistance of Portland Cement Concrete to Scaling Action of Thawing Agents*, by A. G. Timms, PUBLIC ROADS, vol. 28, No. 7, Apr. 1955, pp. 143–157.

(2) *Final Report of Tests of Concrete Containing Portland Blast-Furnace Slag Cement*, by W. E. Grieb and George Werner, PUBLIC ROADS, vol. 31, No. 9, Aug. 1961, pp. 183–193.

(3) *Liquid Membrane-Forming Compound for Curing Concrete, ASTM C 309–58*, in *1961 Book of ASTM Standards, including Tentatives: Part 4, Cement, Lime, Gypsum, Mortar, Concrete, Mineral Aggregates, Bituminous Materials, Soils*, pp. 825–827.

(4) *Evaluation of Air-Entraining Admixtures for Concrete*, by F. H. Jackson and A. G. Timms, PUBLIC ROADS, vol. 27, No. 12, Feb. 1954, pp. 259–267.

Table 14.—Effect of thawing agents on scaling of concrete and rate of thawing of ice in laboratory freezing and thawing tests

Thawing agent[1]	Thawing time,[2] minutes	Rating after freezing and thawing[3] for		
		25 cycles	50 cycles	75 cycles
None	155	1	1	1
Urea, coated	100	5	6	$6\frac{1}{2}$
Urea, uncoated	90	5	6	6
$CaCl_2$	46	$4\frac{1}{2}$	4	$4\frac{1}{2}$
NaCl	51	5	6	$6\frac{1}{2}$
$CaCl_2$ and NaCl	48	5	6	$6\frac{1}{2}$

[1] Thawing agents were used at a rate of 2.4 lb. per square yard of surface.

[2] Time necessary for ice on surface to melt in laboratory air at 70° F.

[3] Each rating is an average of observations for five, 12- by 6- by 2-inch slabs. The slabs were cast on metal bases, were moist cured for 7 days, then were stored in laboratory air for 14 days prior to start of freezing. Air content was 6 percent.

Speed Estimation on Residential Streets

BY THE TRAFFIC OPERATIONS RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

by **RICHARD D. DESROSIERS,**
Highway Research Engineer

The study reported in this article was undertaken to determine with what degree of accuracy residents are able to estimate the normal operating speed of vehicles traveling on suburban streets. This information is expected to be useful to traffic engineers and law enforcement officers in acting upon the many complaints received about speeding cars and trucks in residential areas.

Various factors that might affect the estimations were considered in relation to the type of vehicle for which the speed was being estimated. Conclusions for this study were reached after a statistical analysis of the data collected. However, these conclusions were deemed to apply only to the areas studied because of the limited data collected.

The fieldwork for this study and initial analysis of the data collected were accomplished by a group of Junior Engineers[1] of the Bureau of Public Roads, during 1961, as part of their training.

Introduction

A PROJECT to study the accuracy with which residents estimate the normal operating speeds of vehicles traveling over residential streets was conducted by the Bureau of Public Roads in a selected area in Washington, D.C., in 1961. A passenger car and a panel truck were driven at three controlled speeds of 15, 25, and 35 miles per hour (m.p.h.) and 935 estimates of the speeds at which the vehicles were traveling were made by 160 residents. Generally, the speeds at which the test vehicles were traveling were underestimated. Of the 935 estimates, 42 percent were underestimates, 22 percent were overestimates, and 36 percent were correct estimates, to the nearest 5 m.p.h.

Summary

No general conclusions concerning the accuracy with which speed estimates are made for vehicles on residential streets could be reached because of the limited scope of this study and the data collected. However, the following observations were made: (1) Thirty-eight percent of the estimates of the speed of the passenger car were correct to the nearest 5 m.p.h.; and 31 percent of the estimates of the speed of the panel truck were correct to within 5 m.p.h. (2) More of the observers underestimated the speeds at which both vehicles were traveling than overestimated these speeds. (3) As the travel speeds increased the observers made greater absolute estimating errors in m.p.h. but their percentage error in estimating generally decreased. (4) The age of the observers or the type of vehicle (passenger car or panel truck) had no apparent effect on the mean absolute error made by the observers. (5) Whether the observers had small children seemingly had no effect on the accuracy of their speed estimations, those with and without small children made estimates with a similar degree of accuracy.

In this study no attempt was made to separate the residents who might have complained about speeding traffic, and it is possible that speed estimates from such a group of complainers might provide different results. It is suggested that those conducting future studies of the accuracy of speed estimation might consider using variables such as night interviews; a noisy vehicle; a sportscar; and/or a greater speed range, for example 10 to 50 m.p.h.

Procedure

Two vehicles, a 1959 sedan of a popular make and a panel truck were used as test vehicles for this study. The sedan was chosen because of its size, style, and familiarity to the general public. The truck was a type commonly used for home delivery of bread, milk, and similar commodities. Tests were conducted in which each of the vehicles was being operated at speeds of 15, 25, and 35 m.p.h. These operating speeds were selected because they represent the range of normal operating speeds of vehicles traveling on residential streets. Both the truck and the sedan were operated at each of the speeds so that the estimates for different types of vehicles could be compared.

So that the interviewer would be able to record the actual speed of the test vehicle as it passed the observer, a code system was prearranged in which 6- by 6-inch cards with three different letters of the alphabet on them were used. The driver of the test vehicle selected the speed at random, either 15, 25, or 35 m.p.h., and placed the appropriate card

Table 1.—Mean absolute and percentage errors in speed estimating

Speed	Errors			
	Passenger car		Truck	
	m.p.h.	*Percent*	*m.p.h.*	*Percent*
15	−3.3	−22	−3.4	−23
25	−3.7	−15	−5.0	−20
35	−6.1	−17	−5.4	−15

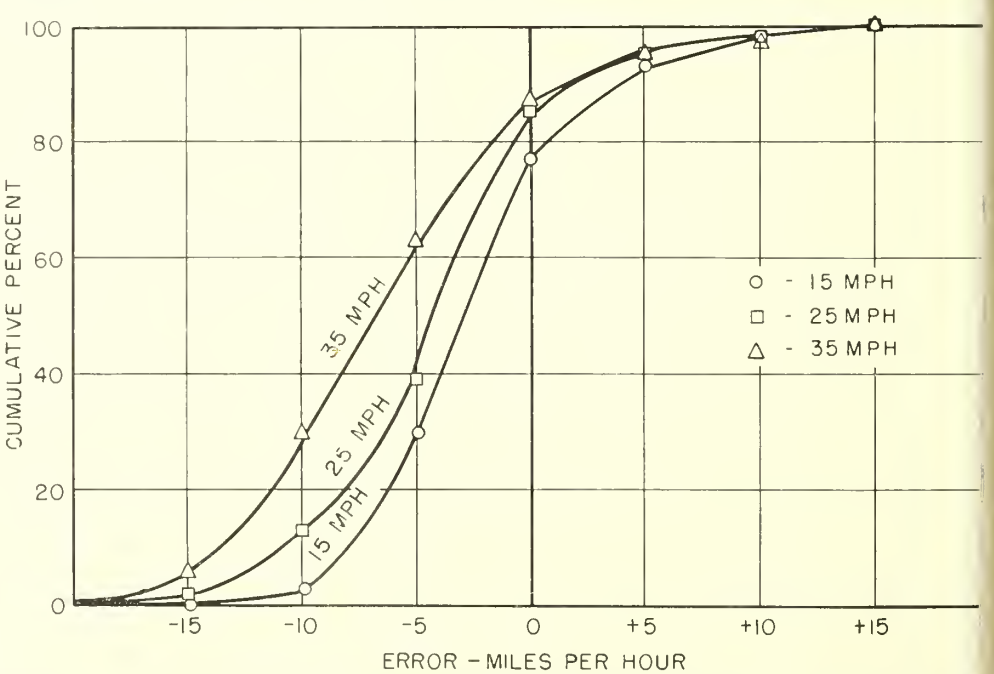

Figure 1.—Cumulative errors in speed estimates for the passenger car at 15, 25, and 35 miles per hour.

[1] The Junior Engineers working on this study were Ben M. Pugh, Lawrence A. Staron, Duane C. Lewis, John H. Girvin, John R. Huff, and William P. Steward.

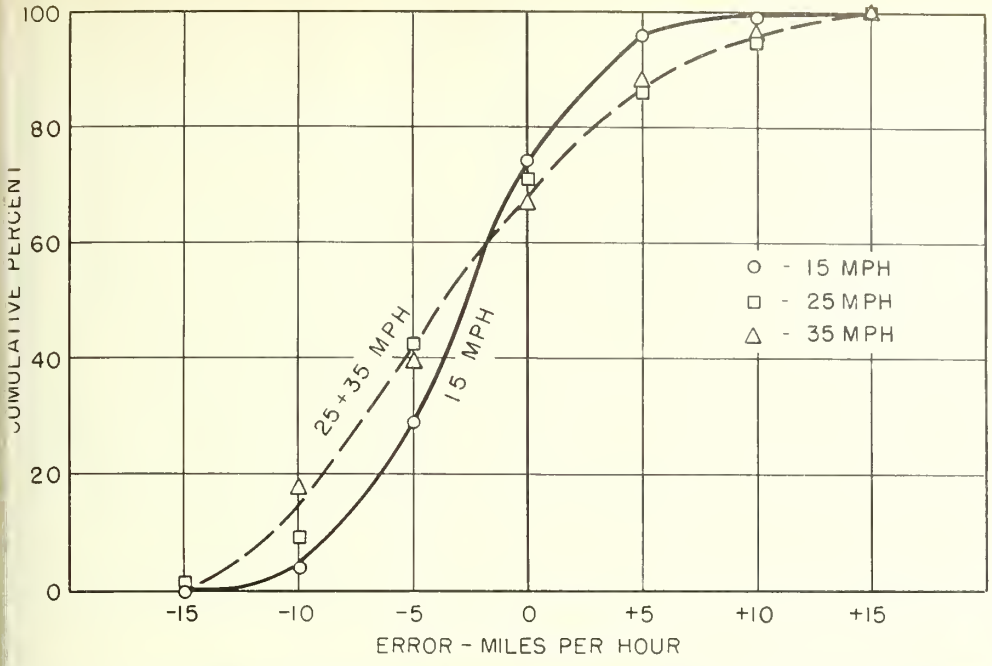

Figure 2.—Cumulative errors in speed estimates for the truck at 15, 25, and 35 miles per hour.

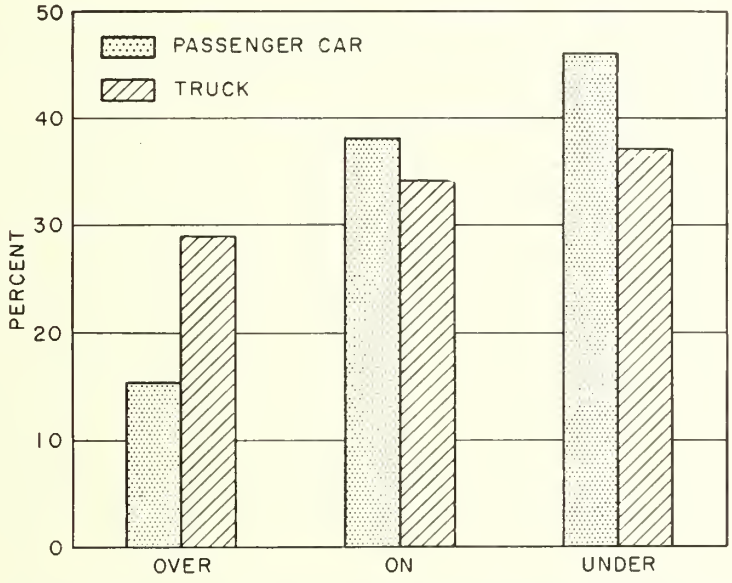

Figure 3.—Percentage of speed estimates over, on, and under for all three test speeds combined.

n the windshield of the test vehicle. Thus he interviewer was able to record the code or the actual speed of the test vehicle at the ame time he recorded the estimate made by he observer, and without giving any clue that night affect the observer's speed estimate. The only pattern followed was that the truck always preceded the passenger car.

Test location

Upper-, middle-, and lower-income neighborhoods of different population densities were selected for the test areas. Within these areas, four residences in each block were selected arbitrarily and the occupants asked to participate in the study. Site selection was limited to locations with the following listed physical characteristics: sufficient sight distance at minor intersections to permit safe crossing, straight-aways long enough for the truck to obtain the required speed, streets wide enough to permit the test without obstructing other traffic, dwelling units situated so that the observers easily could see the vehicles from his or her front door, and turnaround areas for vehicles away from children's play areas. Other factors that might have influenced the speed estimates were recorded by the interviewer: Posted speed limit, parked vehicles, sex and estimated age of the observer, and whether any children 10 years of age or under were in the family.

Observers

Interviewers contacted potential observers and read the following statement: "I am a highway engineer with the U.S. Bureau of Public Roads. We are conducting a Speed Estimation Research Project along this street in front of your house. We would like to engage your cooperation in estimating the speed of our sedan and truck as they pass directly in front of your house."

Those contacted and consenting to participate in the study were given instructions, as follows: "The truck and sedan are marked by large signs on the front stating: 'Speed Research Vehicle.' I will point out the vehicle as it approaches so there will be no identification errors. Each vehicle, the sedan and the truck, will make three passes. Each pass will be at a different speed. We want you to estimate the speed of each vehicle as it passes directly in front of your house, and as you make your estimation I will record it here on this form. We ask only one question other than the speed estimation and that is, 'Do you have any children under 10 years old?'"

A total of 160 observers made 935 individual speed estimates. Most of these estimates were made to the nearest 5 m.p.h. and when they weren't, the interviewer rounded off the estimate to the nearest 5 m.p.h.

Cumulative frequency of errors representing a minimum of 150 observations are shown by the curves in figures 1 and 2. These curves indicate the distribution of errors to have been

Table 2.—Influence of children 10 years of age or under on observers' speed estimates

Observers	Speed estimates—					
	Over		On		Under	
	Car	Truck	Car	Truck	Car	Truck
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
With children 10 or under	16	26	36	35	48	39
Without children 10 or under	17	33	42	33	41	34

Table 3.—Cumulative percentages of correct speed estimates out of a total of 6 estimates per observer

Correct speed estimates	Cumulative percentage of estimates correct to within—		
	2½ m.p.h.	7½ m.p.h.	12½ m.p.h.
	Percent	*Percent*	*Percent*
6	1	44	85
5 or more	6	66	94
4 or more	21	81	98
3 or more	38	92	98
2 or more	66	97	100
1 or more	84	99	100
0 or more	100	100	100

fairly normal for the passenger car for each of three test speeds and for the truck when its speed was 15 m.p.h.; but, the error curves for the truck's speed at 25 and 35 m.p.h. indicate a greater variation in speed estimates. The mean absolute error in miles per hour made by the observers increased as the traveling speed of the vehicles was increased, but the percentage error generally decreased, as is shown in table 1. When all the errors were averaged, the results were negative; that is, the average observer underestimated the speed for all three test speeds for both vehicles. The number of errors made in estimating the speed of the passenger car were nearly identical when its speed was 15 and 25 m.p.h., but the number of errors increased greatly when the speed was increased to 35 m.p.h. As can be seen by figure 3, more underestimates than overestimates were made for the speeds of both vehicles. This was especially true for the passenger car: 46 percent of the estimates were under its speed, and only 16 percent of the estimates were over. Corresponding figures for the errors in estimating the speed of the truck were 37 percent and 29 percent, respectively.

A comparison of the percentage of speed estimates that were over, on, and under the actual speeds is shown for both the car and truck in figure 1. The chi square significance test was applied between the estimates that were over and under the actual speed for each vehicle at each of the three speeds. Significant differences (0.05 level) were obtained for the passenger car estimates when its speed was 25 and 35 m.p.h. No significant differences were obtained for either the passenger car, when its speed was 15 m.p.h., or for the truck at any of the three test speeds. Significance at the 0.05 level means that there are less than five chances in a hundred that the observed differences could have occurred by chance alone.

Prior to this study, it had been thought that people with small children might tend to overestimate the speeds at which vehicles were traveling. It is very evident from the data shown in table 2 that such a hypothesis was not supported by this study. Estimates of observers with small children were similar in accuracy to those of the observers without small children; approximately 65 percent of the observers had small children in their household. Neither did the age of the observer seem to have had an effect on the degree of accuracy. The ages of the observers that had been estimated by the interviewer were divided into ten-year age groupings. Consideration of the speed estimates revealed no apparent differences between these age groups in the absolute error in estimating speed.

Of the 935 estimates, 42 percent were under the speed of the vehicle, 22 percent were overestimates, and 36 percent were correct to the nearest 5 m.p.h. Although those interviewed had not been instructed as to what range should be employed in their estimating, nearly all observers estimated to the nearest 5 m.p.h. If a smaller estimating unit had been used, such as 1 m.p.h., the percentage of correct estimates would have been reduced.

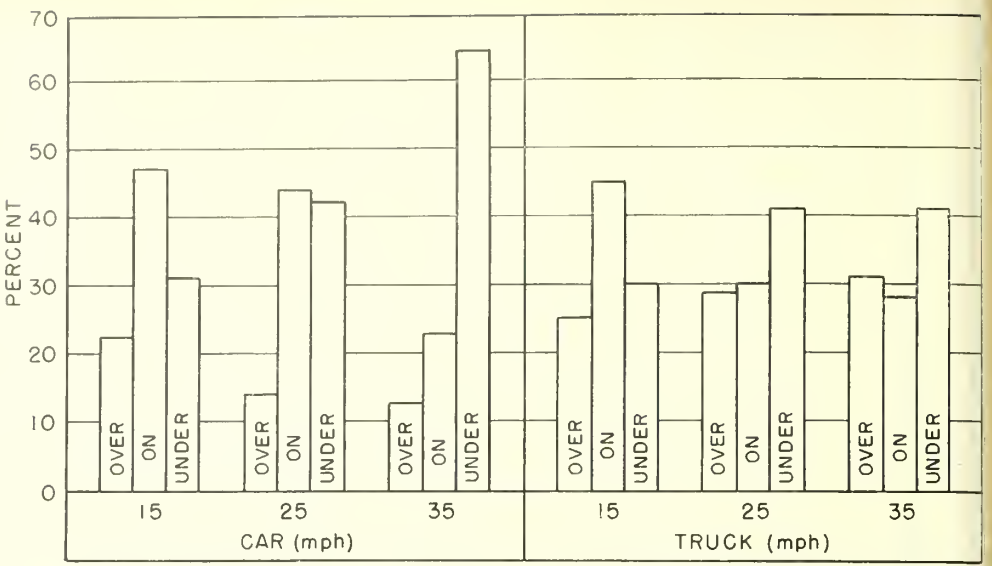

Figure 1.—Percentage of speed estimates over, on, and under for passenger car and truck a each test speed.

With most of the speed observations having been made to the nearest 5 m.p.h., a correct answer was actually 0 plus or minus 2½ m.p.h. Likewise, observation of 5 and 10 m.p.h. errors were actually 5 plus or minus 2½ m.p.h. and 10 plus or minus 2½ m.p.h. The upper (maximum) limits of these errors were used to construct table 3. The data here clearly show that the number of correct observations increased with an increase in the acceptable error, until 85 percent of the observers were never in error by more than 12½ m.p.h. Such figures apply only to the speed range used for this study, 15, 25, and 35 m.p.h., and any extrapolations to higher or lower speeds might lead to erroneous conclusions. The curves shown in figure 5 were constructed from the data of table 3. The use of these curves can be shown best by the following example. In estimating speeds within a 5-mile range, 18 percent of the observers were accurate every time (100 percent) and 38 percent wer accurate 80 percent of the time.

The data collected were further examine to determine whether some individuals alway estimate better than others. If at least fou of the six speed estimates made by an observ were under the correct speed of the vehicl and none of the estimates were over, th subject was considered to have been an under estimator. If at least four speed estimat were over and none were under, the subjec was considered to have been an overestimato If at least four of the estimates were correc the subject was considered to have been correct estimator. On this basis, 34 percen of the observers always underestimated, 1 percent always overestimated, and 20 percen were always correct. The remaining 32 pe cent of the observers could not be placed i any one of these three groups because of th inconsistent pattern of their estimating.

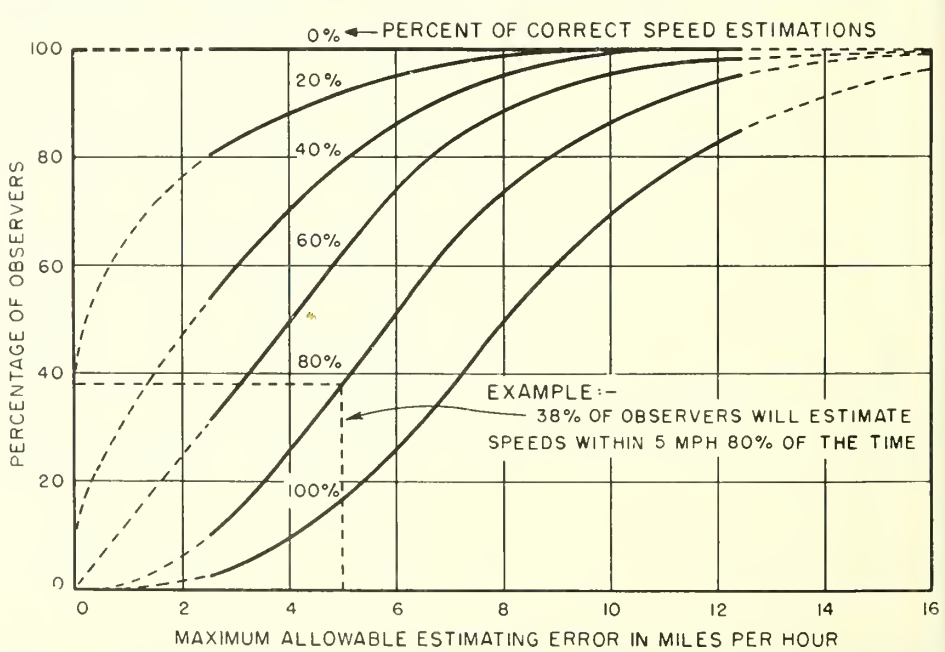

Figure 5.—Percentage of correct speed estimates as a function of the maximum allowab estimating error, and the percentage of observers.

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Correlation of Compaction and Classification Test Data of Soils

BY THE OFFICE OF RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Reported[1] by GEORGE W. RING, III, and JOHN R. SALLBERG, Highway Research Engineers, Physical Research Division, and WEBSTER H. COLLINS, Bridge Engineer, Development Division

This article presents the results of two studies undertaken by the Physical Research Division, Bureau of Public Roads, to correlate the results of laboratory compaction tests with the results of classification tests. The correlations developed in these studies have been proved useful for rapidly checking compaction test results and for denoting unusual soil characteristics that might cause construction difficulties.

Laboratory compaction tests are employed to provide data on maximum dry density and optimum moisture content of soils; results of such tests are used widely in the development of requirements for earthwork compaction. Classification tests consist of grain-size analysis and the determination of plastic and liquid limits. In the two correlation studies described here, the compaction tests were performed in accordance with an AASHO test method; the grain-size analyses were extended to include the 0.001 millimeter size; and the moisture contents, denoting plastic and liquid limits of the soil, were determined in the standard manner.

The first study, to correlate compaction and classification test results was conducted mainly by plotting maximum dry densities and optimum moisture contents against plastic and liquid limits to arithmetic scale. In the second study, multiple linear regression analyses were used; this permitted correlation of compaction test results with several characteristics of the grain-size analysis, as well as with the plastic and liquid limits.

The formulas developed during these studies, incorporating the various factors for estimating compaction tests results, appear to be more reliable for a wide variety of soils than any previously published.

Introduction

AMONG the soil tests required for controlling the quality of highway construction, the compaction test is one of the most important and one of the most time-consuming. A need exists for (1) shortening the time required to perform laboratory compaction tests, and (2) developing interrelationships between compaction test data and other laboratory test data to increase the basic knowledge of compaction. The results of two studies conducted by the Bureau of Public Roads to accomplish these two objectives are reported in this article.

Summary

The optimum moisture content and maximum dry density obtained in compaction tests are known generally to be related to the plasticity and gradation of the soil material. This article shows the relationships of optimum moisture content and maximum dry density—as determined in the test, *The Moisture-Density Relations of Soils Using a 5.5-lb Rammer and a 12-in. Drop* (AASHO Designation: T 99–57, Method A)—to each of several plasticity and gradation characteristics and to two or more plasticity and gradation characteristics jointly.

The multiple correlations, based on two or more plasticity or gradation characteristics, provided several methods for predicting optimum moisture content and maximum dry density. The simplest of these prediction methods seems to be better than those previously published when a variety of soils from large geographical areas are being considered. The best correlations of compaction data with classification data were obtained when the analyses were made on test data from only one county, which was the smallest geographical unit considered.

Predicted optimum moisture content and maximum dry density are useful for several purposes: (1) determining the amount of water to use in the compaction test for the first moisture-density point; (2) rapidly appraising compaction test results when the classification data are readily available; (3) reducing the number of compaction tests required in areas where the prediction methods have been proved to be sufficiently accurate for construction control purposes, and (4) denoting unusual soils that are different from those generally encountered and that may cause construction difficulties.

Published Correlations

In 1938, Woods and Litehiser (*1*)[2] showed the general interrelation of plastic limit, plasticity index, liquid limit, optimum moisture content, and maximum dry density for 1,367 Ohio soils. They reported that increases in the plastic properties of the soils were accompanied by increases in optimum moisture content and decreases in maximum dry density. In a more recent report, Jumikis (*2*) presented a chart relating optimum moisture content obtained from ". . . standard soil compaction tests . . ." with liquid limit and plasticity index for various New Jersey glacial soils. This chart is shown in figure 1.

Rowan and Graham (*3*) presented two formulas for estimating the maximum dry density and the optimum moisture content as determined in the Proctor test.[3] These formulas are, as follows:

---

[1] Presented at the 41st annual meeting of the Highway Research Board, Washington, D.C., January 1962.

[2] References indicated by italic numbers in parentheses are listed on page 86.

[3] The details of the compaction test used by Rowan and Graham were not given. In the original Proctor test (*4*), 25 firm 12-in. strokes of a 5.5-lb tamper were used on each of three layers of soil in a mold about 4 inches in diameter and 5 inches in height.

Table 1.—Summary of deviations of optimum moisture contents estimated by the PL and LL chart (fig. 1) from those determined by test

State	Predominant soil type (origin)[1]	No. of samples for which the estimated optimum moisture content was less than the test result by amount indicated											No. of samples for which the estimated optimum moisture content exceeded the test result by amount indicated					
		−11	−10	−9	−8	−7	−6	−5	−4	−3	−2	−1	0	+1	+2	+3	+4	+5	+6
Alabama	Residual											3	16	12	1				
Arizona	do						1	1	2	1			2					1	
Arkansas	Recent alluvium								1	1	2	4	10	8	2				
Connecticut	Glacial								1	1	1	1	7	2	2	1			
Florida	Coastal plain sand									1	1	1	3	4	1				
Idaho	Non-soil	1	1	2			4	5	4	4	6	11	20	9	2		3		1
Illinois	Loessial										1	10	12	5					
Kentucky	Residual									1		6	18	4	1				
Maryland	do										2	3	6	7	3	1	2		
Minnesota	Glacial								2	4	4	4	3	2	1				
Nebraska	Outwash								1	1	7	7	7	15	7	4	1	1	
Nebraska	Loessial										2	4	13	8		1			
Nevada	Non-soil			1	1	2	1	1	2	1									
New Mexico	Residual										1	2	4	1	2				
North Carolina	do										1	3	5	5	1	2		1	
North Dakota	Glacial							1	2	2	2	2	4	4	2				
Oregon	Non-soil					1	2	2	1	1	1	2	7		3				
Tennessee	Residual							1	3		10	11	7	6				1	
Texas	Coastal plain clay											2	7	1	2		1		
Texas	Residual								1	1	2			1	2				
Vermont	Glacial							1		1		3	4	7					
Ohio	Lacustrine								1		1	1	2	2					
Totals		1	1	3	1	3	8	13	21	28	46	92	160	87	30	5	7	3	1

[1] See reference *10*.

Calculated density, pounds per cubic foot[4] =

$$\frac{D}{1+\frac{D-C}{62.5\,G_s}} \quad \ldots\ldots\ldots\ldots (1)$$

Calculated optimum moisture, percent =

$$SL\left(\frac{B}{A}\right) \quad \ldots\ldots\ldots\ldots (2)$$

Where,

$D = \frac{CA}{B}$

C = 62.5 · shrinkage ratio, p.c.f.

A = percentage passing No. 4 sieve

B = percentage passing No. 40 sieve

G_s = specific gravity

SL = shrinkage limit, percent.

In comparing the predicted maximum dry densities and optimum moisture contents with test results for 10 soils, Rowan and Graham (*3*) found that the greatest difference between the predicted and the test maximum dry densities was about 5 percent, and that the predicted optimum moisture contents were slightly higher, from about 1 to 5 percentage points, than the test results. They suggested that all calculated (predicted) optimum moisture contents be corrected by subtracting 3 percentage points.

Davidson and Gardiner (*5*) used the Rowan-Graham formulas in an analysis of test data for 240 soils from 11 States. They

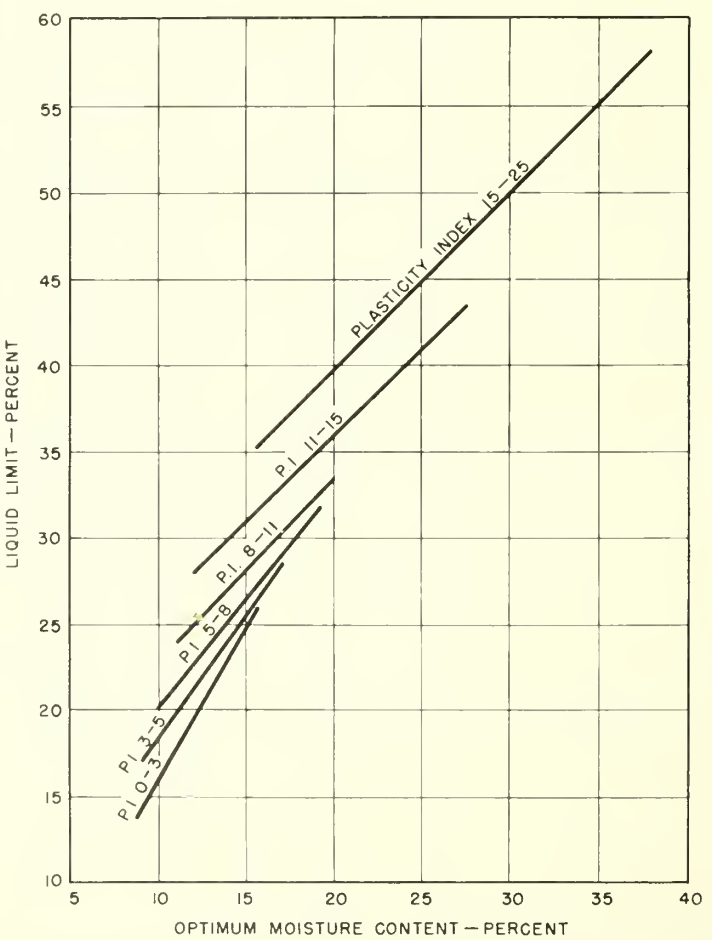

Figure 1.—Optimum moisture content and liquid limit related to plasticity index of various glacial materials (2).

[4] Pounds per cubic foot hereafter indicated by p.c.f.

Table 2.—Comparisons of optimum moisture contents and maximum dry densities estimated by PL and LL chart (fig. 1) with those determined by test[1] for soils from Alaska and outside the continental United States

Soil		Optimum moisture content			Maximum dry density		
Location	Kind	Test	Estimated	Deviation	Test	Estimated	Deviation
		Percent	*Percent*	*Percent*	*P.c.f.*	*P.c.f.*	*P.c.f.*
Alajuela, Costa Rica	Lateritic	41	22	−19	77	99	+22
Las Lomas, Panama	do	29	19	−10	88	104	+16
Liberia	do	20	24	+ 4	107	95	−12
Do	do	12	14	+ 2	126	114	−12
Do	do	15	17	+ 2	117	108	− 9
Addis Ababa, Ethiopia	Black Cotton	39	29	−10	80	87	+ 7
Oahu, Hawaii	Kaolinitic	33	22	−11	85	99	+14
Do	do	36	23	−13	80	97	+17
Do	Montmorillonitic	37	30	− 7	79	86	+ 7
Do	do	35	27	− 8	84	90	+ 6
Kodiak, Alaska	Silty alluvium	20	21	+ 1	103	100	− 3
Do	do	24	23	− 1	95	97	+ 2
Do	do	33	30	− 3	80	86	+ 6
Kenai-Kasilof, Alaska	do	10	10	0	131	125	− 6
Do	do	14	14	0	118	114	− 4
Do	do	13	12	− 1	124	119	− 5

[1] Data from Public Roads laboratory except for Hawaiian soils, which are from Kawana and Homes (*11*).

found wide deviations between the predicted and test results obtained from ". . . the standard Proctor control tests . . ." on highly plastic soils. They determined that the size of the deviation was related to the plasticity index of the soil and revised the Rowan-Graham formulas to fit these data more closely. The revised formulas are, as follows:

Calculated density, p.c.f. =

$$\frac{6250\ K_1}{SL\left(\frac{B}{A}-1\right)+\frac{100}{R}} \quad \ldots\ldots\ldots (3)$$

Calculated optimum moisture, percent =

$$SL\left(\frac{B}{A}\right)+K_2 \quad \ldots\ldots\ldots\ldots (4)$$

Where,

$K_1=\frac{312-2(PI)}{300}$

PI = plasticity index

R = shrinkage ratio

$K_2=\frac{PI}{3}-4.$

Formula (3) does not include the specific gravity term (G_s), which appears in formula (1); Rowan and Graham calculated specific gravity from shrinkage test data, whereas Davidson and Gardiner substituted the shrinkage data directly into the formula.

Turnbull (*6*) of Australia showed that the optimum moisture content is closely related to the gradation of the soil. For a numerical measure of gradation, he used the area above the graph of the grain-size distribution curve and named it the classification area (*7*). The solid, curved line in figure 2 shows the relationship of classification area to optimum moisture content for 101 soils tested by a compaction method very similar to the AASHO T 99–57, Method A. (The compaction effort that Turnbull used was 15 percent greater than the AASHO effort.) The simple curvilinear relationship shown in figure 2 fits the test data for optimum moisture content very closely; 72 percent of the predicted optimum moisture contents were within 1.0 percentage point of the test results. Additional tests made later by Turnbull, on coarse-grained soils with high classification areas, indicated that the curve in figure 2 should be corrected to follow the dotted line shown in the figure. Information on these tests was transmitted to the authors by a letter from Mr. Turnbull dated Oct. 10, 1961.

To simplify the determination of the classification area, Turnbull subdivided the grain-size distribution chart by equally spaced ordinates; figure 3 is an adaptation of Turnbull's chart. The original chart by Turnbull extends to the right five more units to include particle sizes up to 6 inches. The extra five

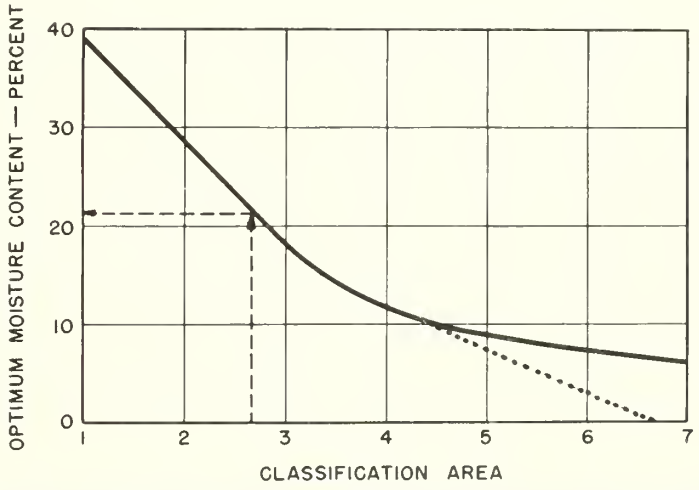

Figure 2.—Optimum moisture content vs. classification area based on test data from 101 soils (6). The dotted line shows position of corrected curve based on additional tests of coarse material.

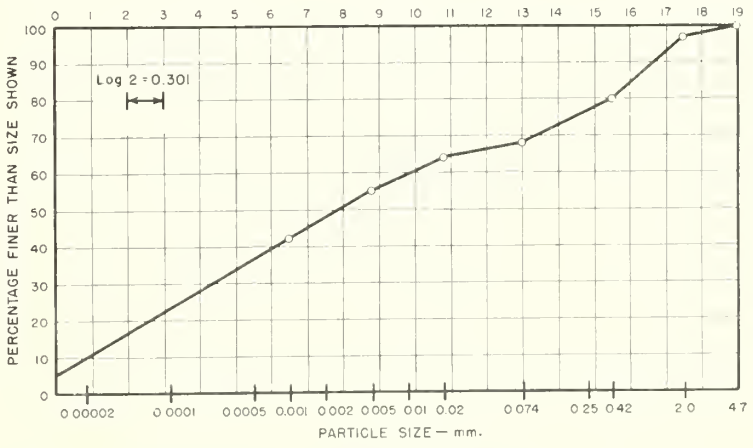

Figure 3.—Grain-size distribution of Cecil coarse sandy loam on a chart for determining Turnbull classification area (7).

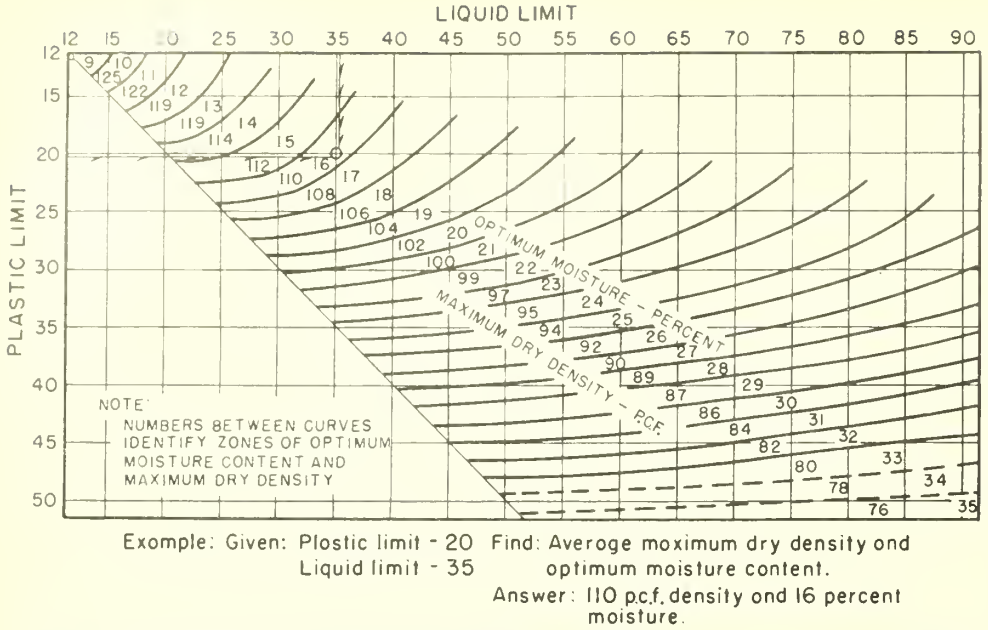
LIQUID LIMIT
PLASTIC LIMIT
OPTIMUM MOISTURE - PERCENT
MAXIMUM DRY DENSITY - P.C.F.
NOTE:
NUMBERS BETWEEN CURVES IDENTIFY ZONES OF OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY
Example: Given: Plastic limit - 20
Liquid limit - 35
Find: Average maximum dry density and optimum moisture content.
Answer: 110 p.c.f. density and 16 percent moisture.

Figure 1.—Relation of average maximum dry density and optimum moisture content to plastic limit and liquid limit.

units were not needed in the Public Roads study because the maximum size of particles used in the compaction test was 4.7 mm. (about three-sixteenths of an inch).

To determine the classification area from the grain-size distribution curve of a given soil material, one-half of the length in percent of ordinate zero above the curve should be added to the sum of the lengths of ordinates 1 through 19 above the curve, and that sum should be multiplied by 0.00301. For example, the gradation of a sample of Cecil coarse sandy loam has been plotted on figure 3. The length of the ordinates to be added above the curve are $\frac{95}{2}$, 89, 84, 78, 72, 67, 61, 55, 50, 44, 40, 36, 34, 32, 27, 23, 16, 8, 2, and 0. The sum of these, 866, when multiplied by 0.00301 yields a classification area of 2.61. By locating the point corresponding to this area in figure 2, the optimum moisture content is determined to be 22 percent.

FIRST PUBLIC ROADS STUDY

The first attempt, in 1958, by the Bureau of Public Roads to correlate optimum moisture content and maximum dry density with classification data was based on test data of 972 soil samples from 31 States. The compaction test used in the first study, as well as in the second, was performed in accordance with AASHO Designation: T 99–49, which is the same as the current AASHO Designation: T 99–57, Method A. In this method, the soil fraction passing the No. 4 sieve is compacted in three layers in a 4-in. diameter mold by dropping a 5.5-lb. rammer from a height of 12 inches, 25 times per layer.

Correlations were made by plotting the test data on rectangular coordinates. The chart shown in figure 4, developed by Yemington (*8*) in the first study, correlates optimum moisture content and maximum dry density with plastic limit and liquid limit. To evaluate the chart, it was used to estimate optimum moisture contents for 510 additional soil samples from a number of States. These estimates were compared to test results; these comparisons are given in table 1. The comparisons show that 81 percent of the predicted optimum moisture contents were within 2 percentage points of the test optimum moisture contents. The correlation was best for eastern soils; 94 percent of the predicted optimum moisture contents for the 222 samples from east of the Mississippi River were within 2 percentage points of the test results. Soils showing the least correlation were from non-soil areas west of the Mississippi River. To evaluate estimates of maximum dry density from the chart (fig. 4), a study was made of test data from 532 samples, which included the 510. Sixty-three percent of the estimates were within 4 p.c.f. of the corresponding test results.

Another appraisal of the chart was made with test data for soil samples obtained from Alaska and places outside the continental United States. The comparisons, given in table 2, ranged from reasonably good for soils from Alaska to extremely poor for those from Costa Rica, Panama, and Hawaii. Whereas, the data clearly show that wide variations occur between the predicted and the actual test results of optimum moisture contents and maximum dry densities for certain soils, the variety of soils studies was too limited to warrant conclusions as to the cause of the variations.

The results of Public Roads' first study proved to be very useful. The chart, figure 4, has been used for several years as a guide in performing compaction tests, particularly to estimate the amount of water to use fo the first moisture-density point. It also ha been used by the laboratory supervisor to determine whether the optimum moisture contents obtained by technicians were reasonable. The use of the chart is limited, however, in that it does not fit a large number of unusual soils. Further analysis was needed to make the correlations applicable to a wider range of soils and to make the estimates more accurate.

Table 3.—Correlation between variables as determined by inspection of data plotted on rectangular coordinates

Correlation	Rating [1] of correlation			Relationship on arithmetic scale		Relationship on log log scale	
	Good	Fair	Poor	Linear	Curved	Linear	Curved
Optimum moisture content versus:							
LL	X			X		X	
PL	X			X		X	
PI		X		X			X
Range			X				
D'_{50}		X			X	X	
FA		X			X		X
0.001		X		X			X
Maximum dry density versus:							
O.M.C.	X				X		X
LL		X		X			X
PL	X			X			X
PI		X					
Range			X				
D'_{50}		X					
FA		X			X		X
0.001		X					

Legend:
LL, Liquid limit.
PL, Plastic limit.
PI, Plasticity index.
Range, Number of log cycles traversed by the straight line approximating the D_{10} to D_{90} portion of the grain-size distribution curve.
D'_{50}, The average particle size, determined at the midpoint of the straight line referred to in Range definition.
FA, Fineness average, equal to one-sixth of the sum of percentages of particles finer than the following sizes in millimeters: 2.0, 0.42, 0.074, 0.020, 0.005 and 0.001.
0.001, Percentage of particles finer than 0.001 millimeter.

[1] Ratings were based on the degree of scatter of the plotted points about the line of best fit.

SECOND PUBLIC ROADS STUDY

To improve the methods for predicting optimum moisture content and maximum dry density, Public Roads made a second study in 1961 using multiple linear regression analysis. This method of analysis permitted all of several variables to be used jointly for estimating optimum moisture content and maximum dry density.

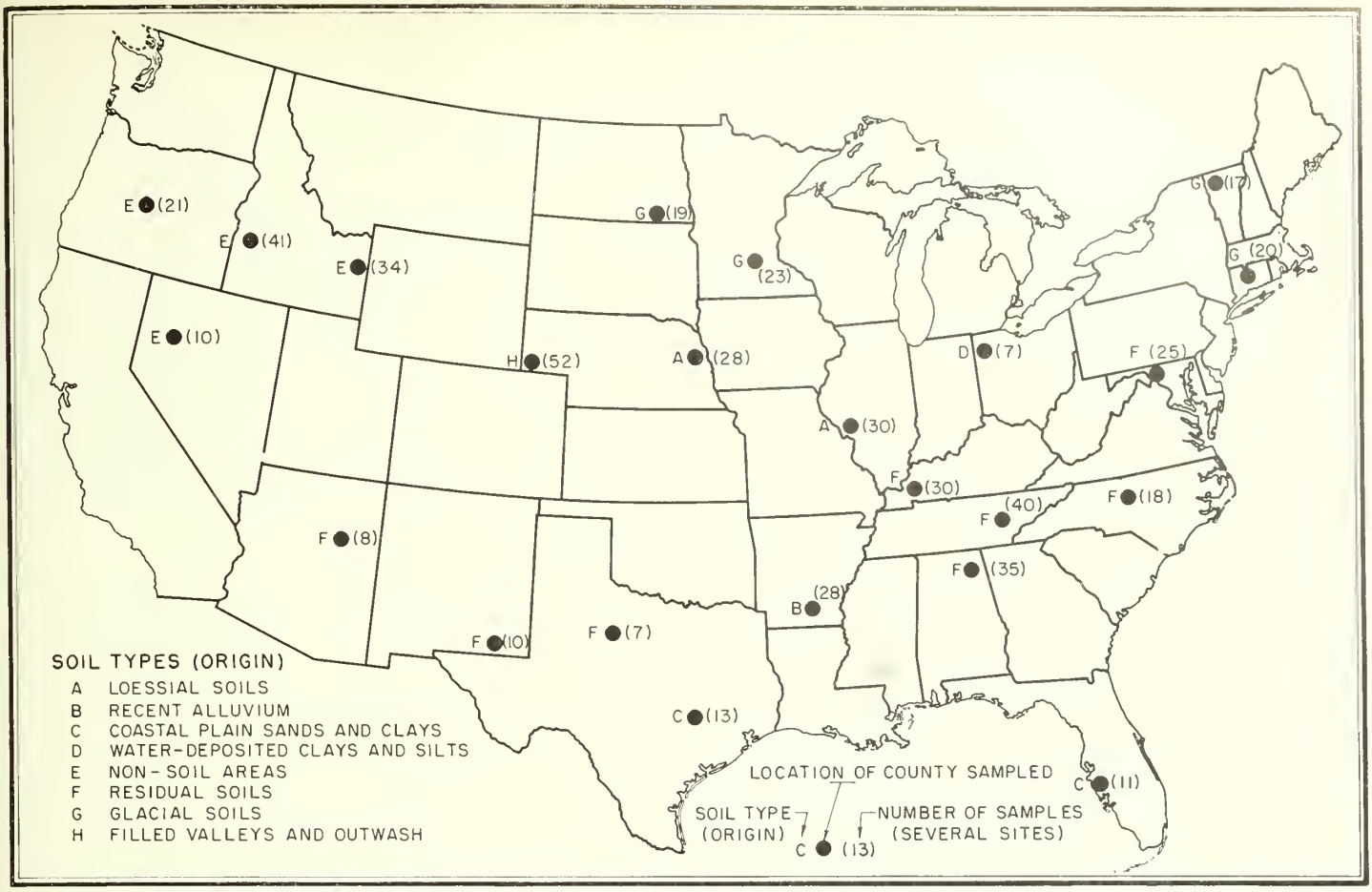

Figure 5.—Primary soil type (origin), location of county samples, and number of samples.

Selection of Samples and Variables

Soil test data, to represent a broad coverage of soils within the continental United States, were selected from the files on the basis of the geographical and geological origin of the soil samples. Initially, 946 samples were selected; many were the same as those used in the first study. This number was reduced to 600 by the use of a set of random numbers. The nonplastic soils were eliminated after preliminary analyses of the test data had shown them to have different interrelationships from those of the plastic soils. The analyses were made on the remaining 527 samples of plastic soil. The general types (origins) of soils represented, their sampling locations, and the number of samples from each location are shown in figure 5.

The independent variables used in the analyses included plastic limit, liquid limit, plasticity index, and several measures of gradation. Specific gravity also was considered but was not used because of insufficient data. Standard AASHO tests were used in determining the plasticity and gradation of the soils; gradation was determined on the fraction passing the No. 4 sieve because that was the fraction used in the compaction test, AASHO T 99–57, Method A. Gradation, as represented by percentages passing specific sieves, could not be used as an independent variable because the regression type of analysis requires that each variable be expressible or measurable by a single number. To represent characteristics of gradation, several measures were tried. Burmister (*9*) reported, "The significant characteristics of grain-size distribution are fineness, range of grain size, and type of grading." All of these characteristics were evaluated in the second Public Roads study.

Three measures were devised for fineness, as follows: (1) Percentage of particles finer than the 0.001-mm. size was designated as 0.001 fraction. (2) Fineness average, FA, was determined by taking one-sixth of the total of the percentages of particles, by weight, finer than the following listed sizes in millimeters: 2.0 (No. 10), 0.42 (No. 40), 0.074

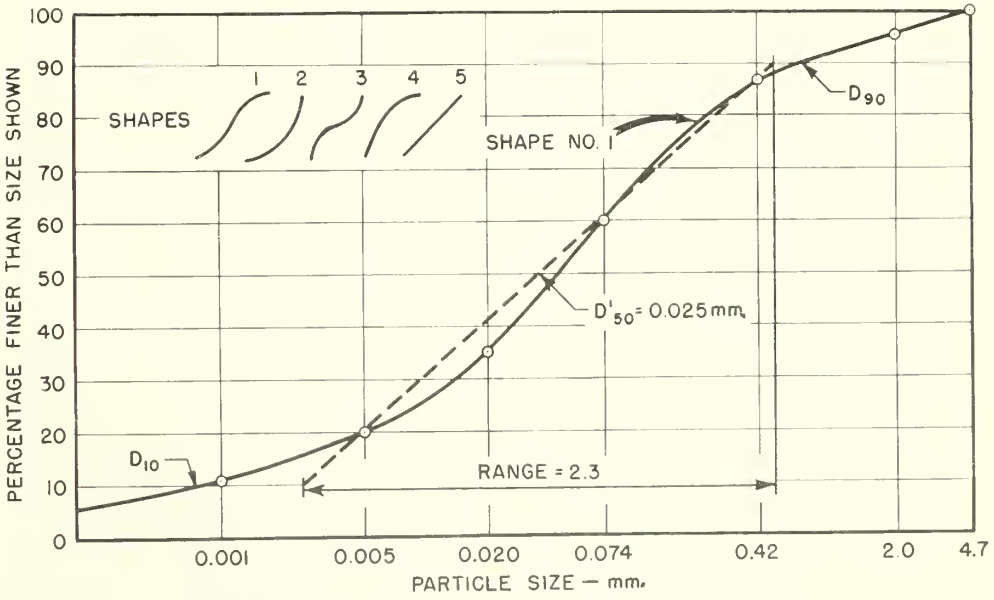

Figure 6.—Typical grain-size distribution curve showing shape, D'_{50}, and range.

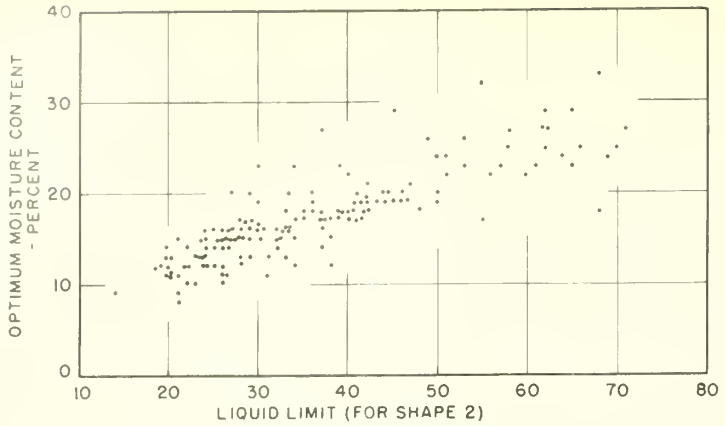

Figure 7.—A preliminary plot of optimum moisture content vs. liquid limit.

(No. 200), 0.020, 0.005, and 0.001. (3) The average particle size, D'_{50}, was obtained at the midpoint of a straight line (fig. 6) drawn to approximate the major portion of the grain-size distribution curve. The dashed line in figure 6 is an example of this straight line; it approximates the grain-size distribution curve from D_{10} to D_{90}. The percentage finer than the 0.001-mm. size was used in the first two measures mainly because it is the finest fraction normally measured in the Public Roads soils laboratory. The range of grain or particle sizes, designated herein as range, is defined as the number of log cycles traversed by the straight line approximating the D_{10} to the D_{90} portion of the curve. The type of grain-size distribution curve was designated shape. The five shapes considered are shown in figure 6.

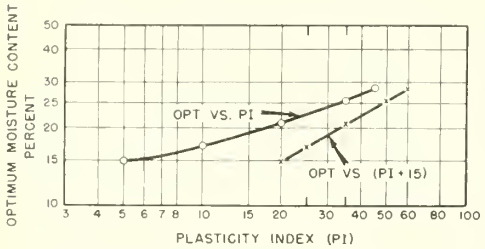

Figure 8.—Logarithmic plots of optimum moisture content vs. PI, and optimum vs. (PI+15), showing effect of adding a constant to the independent variable.

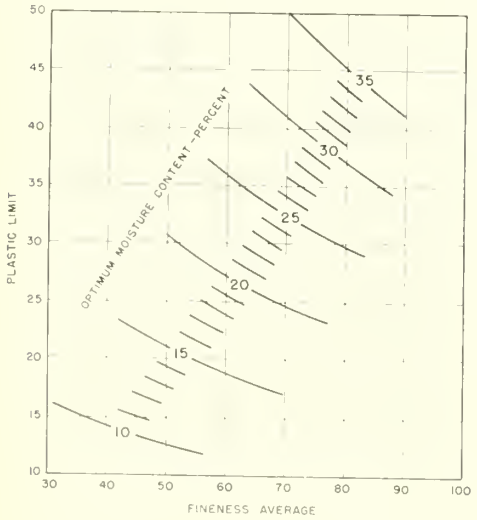

Figure 9.—Relation of optimum moisture content to plastic limit and fineness average, analysis No. 4.

Study of Simple Relationships

To examine the simple relationships of each dependent variable—optimum moisture content and maximum dry density—with each independent variable, plots were made on rectangular coordinates to arithmetic scale. Separate plots were made for each shape. An example, optimum moisture content versus liquid limit for shape 2, is given in figure 7. The results indicated good correlations of optimum moisture content with liquid limit and with plastic limit, and good correlations of maximum dry density with optimum moisture content and with plastic limit. A summary of the findings is given in table 3. Whenever a definite linear or curvilinear relation was developed on arithmetic scale, that line approximating the data was replotted to logarithmic scale. The type of relationship resulting, linear or curvilinear, also is shown in table 3. An examination of the plotted data for each shape indicated that separating the data on the basis of shape was of little or no value; hence, the data for all the samples were subsequently analyzed together regardless of shape.

Regression Analysis by Electronic Computer

The multiple linear regression analyses were performed on an IBM 650 electronic computer using a computer program (file number 6.0.001) supplied by the machine manufacturer. The multiple regression analysis is a method for obtaining a formula for estimating one variable by means of several other variables. The analysis provides the linear equation that best fits the data. The results of the multiple linear regression analyses are summarized in table 4. The variables and the standard error of estimate[5] are given for each analysis. The formulas that were developed are listed in table 5.

●In the first of five regression analyses, to determine equations for predicting optimum moisture content, a relationship was sought using all six independent variables. The analysis was made without adjustments for the curvilinear relationships of D'_{50} and FA (table 4). The resulting standard error of estimate was ±2.00 percent moisture.

●In analysis No. 2, logarithmic transformations were made of all variables in order to make the linear program applicable to curvilinear relationships. The resulting standard error of estimate indicated a slightly poorer correlation than that obtained in analysis No. 1. It was found that the logarithmic transformation, in addition to straightening out certain curvilinear relationships, had caused some linear relationships to become curvilinear.

[5] The standard error of estimate is a measurement of deviation or degree of scatter of the points (test results) around the regression line. If the normal distribution of error holds, 67 percent of the test results will be within one standard error of the predicted result, and 95 percent will be within 2 standard errors. The unit of measure is the same as that of the predicted variable.

Table 4.—Variables and resulting standard error of estimate for each regression analysis

Analysis No.	Dependent variable	Independent variables [1]							Standard error of estimate	Remarks
		O.M.C.	LL[4]	PL	PI	D'_{50}	FA	0.001		
1	O.M.C	------	X	X	X	X	X	X	[2] ±2.00	Data used directly.
2	---do---	------	X	X	X	X	X	X	±2.05	Log transformation. [3]
3	---do---	------	X	X	X	X	X	X	±1.98	Log transformation with adjustments. [4]
4	---do---	------	----	X	----	----	X	------	±2.17	Do.
5	---do---	------	----	X	----	----	X	------	±1.13	Do. [5]
6	M.D.D.	------	X	X	----	----	X	------	±4.44	Data used directly.
7	---do---	X	X	X	----	----	X	------	±2.52	Log transformation with adjustments. [4]
8	---do---	------	----	X	----	----	X	------	±4.32	Do.
9	---do---	------	----	X	----	----	X	------	±2.98	Do. [5]

[1] Independent variables are simplified; see table 5 for exact form of each variable. Meanings of abbreviations are given in table 3.
[2] The standard error of estimate by predicting with PL alone was ±2.45; the predicting equation is O.M.C.=0.811 PL+0.430.
[3] All variables were transformed to natural logarithms.
[4] Constants were added to some independent variables before logarithmic transformation to make all relationships linear with dependent variable.
[5] Analysis is for 40 samples from Loudon Co., Tenn.

Table 5.—Summary of predicting formulas from Public Roads Study No. 2

Analysis No.	Predicting formula
1	$O.M.C.=1.427\ LL-0.815\ PL-1.373\ PI-0.0007\ D'_{50}+0.062\ FA+0.035\ (0.001\ fraction)-1.312$
2	$\text{Log}\ O.M.C.=0.158\log(\frac{LL}{10})+0.647\log PL+0.021\log PI+0.012\log(\frac{D'_{50}}{100})+0.354\log FA+0.248\log(0.001\ fraction)-0.974$
3	$\text{Log}\ O.M.C.=1.029\log LL+0.045\log PL+0.224\log(PI+15)-0.033\log(\frac{D'_{50}}{100})+0.229\log(FA+100)+0.098\log(0.001\ fraction+40)-3.401$
4	$\text{Log}\ O.M.C.=0.784\log PL+1.378\log(FA+100)-6.586$
[1] 5	$\text{Log}\ O.M.C.=0.763\log PL+1.377\log(FA+100)-6.544$
6	$M.D.D.=147.525-0.020\ LL-1.195\ PL-0.198\ FA$
7	$\text{Log}\ M.D.D.=7.126-0.653\log(OMC+15)+0.059\log LL-0.120\log(PL+20)+0.014\log FA$
8	$\text{Log}\ M.D.D.=7.247-0.567\log(PL+20)-0.110\log FA$
[1] 9	$\text{Log}\ M.D.D.=7.105-0.518\log(PL+20)-0.113\log FA$

Legend:
O.M.C., Optimum moisture content.
M.D.D., Maximum dry density.
LL, Liquid limit.
PL, Plastic limit.
PI, Plasticity index.
D'_{50}, The average particle size on a straight line approximating the grain-size distribution curve.
FA, Fineness average.
0.001 fraction, Percentage of particles finer than 0.001 millimeter.
log, Natural logarithm.

[1] For 40 samples from Loudon Co., Tenn.

●Prior to analysis No. 3, the average linear and curvilinear relationships developed in arithmetic plots of the data were plotted to logarithmic scale to determine the effect of logarithmic transformation on all the variables. When the resulting plot was curved, constants were added to the independent variables to obtain straightline relationships. An example of this type of adjustment is shown in figure 8. The constants were determined by trial and error. The standard error of estimate of ±1.98 percent moisture indicated a slightly better relationship than found in the previous analysis.

●In analysis No. 4, optimum moisture content was compared with only two independent variables—plastic limit and fineness average. The number of variables was reduced to simplify the predicting equation. Plastic limit and fineness average were used because the partial correlation coefficients from previous analyses had indicated them to be the best two of the independent variables for predicting optimum moisture content. The reduction of independent variables, however, reduced the accuracy of the predicting formula; the standard error of estimate increased to ±2.17 percent moisture. The predicting formula developed in this analysis was:

$$\log O.M.C.=0.784\log PL+1.378\log(FA+100)-6.586 \quad \ldots\ldots\ldots (5)$$

Where,

log = natural logarithm
O.M.C. = optimum moisture content, percent
PL = plastic limit
FA = fineness average.

Figure 9, developed from formula (5), shows optimum moisture contents for the range of plastic limits and fineness averages studied.

The standard errors of estimate shown in table 4 were based on the numerical differences between the measured and predicted optimum moisture contents and maximum dry densities. These standard errors of estimate may give the erroneous impression that the optimum moisture content or the maximum dry density can be predicted with the same accuracy through a wide range of values. A more realistic picture of the relationship between predicted optimum moisture content from formula (5) and the test optimum moisture content is shown in figure 10. The deviations, in percent moisture, increased as the actual optimum moisture content increased.

●Analysis No. 5 was made with data representing 40 samples from Loudon Co., Tenn. The test data from a single county were

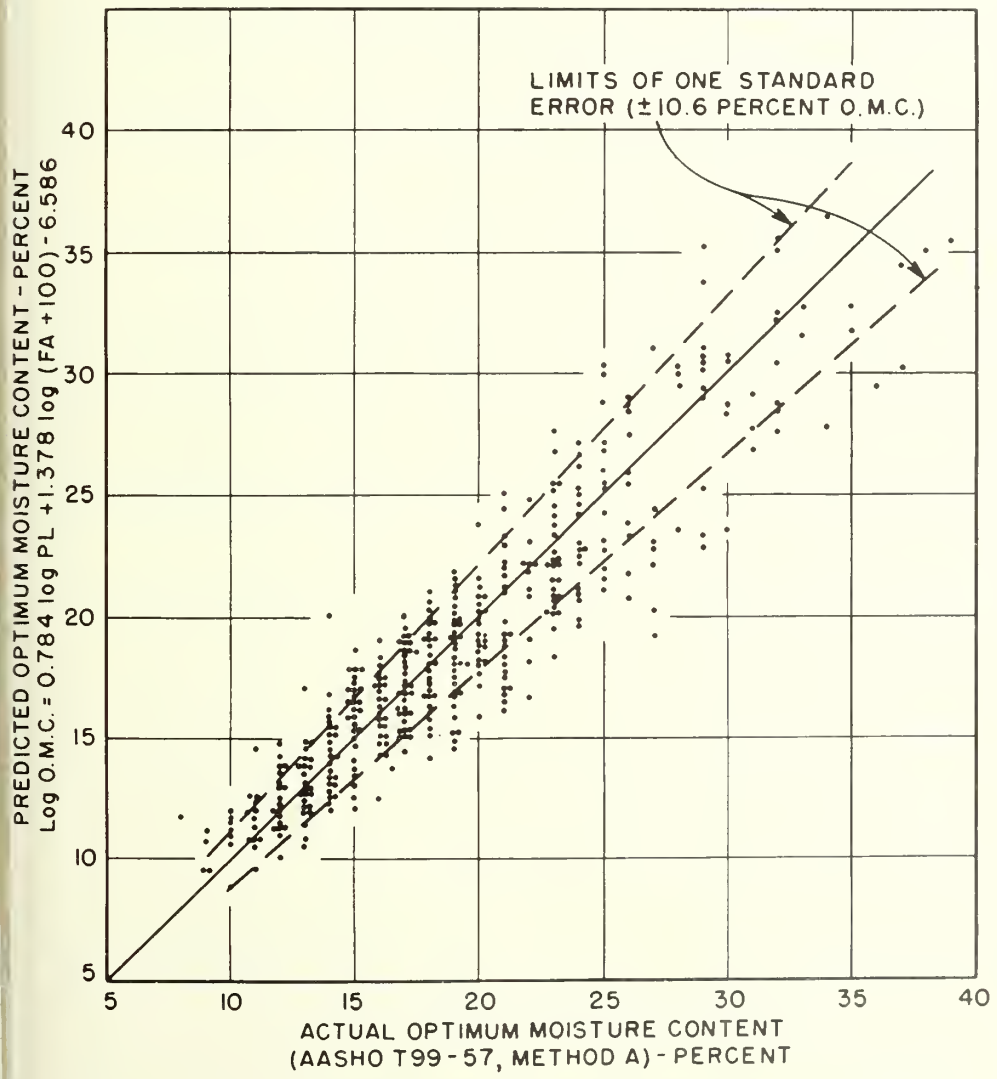

Figure 10.—Actual vs. predicted optimum moisture content from analysis No. 4.

Table 6.—Soil data used in comparison of methods for predicting optimum moisture content

Location sampled	Soil [1] series	Horizon sampled	Optimum moisture content	Maximum dry density	LL	PL	PI	Specific gravity	Shrinkage limit	Shrinkage ratio	Gradation finer than sizes shown in millimeters [2]							FA
											4.7	2.0	0.42	0.074	0.020	0.005	0.001	
			Percent	*P.c.f.*							*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	
Ottawa, Ill	(3)		14	119	28	15	13	2.72	13	1.95	100	98	92	78	62	41	21	65
Bayfield Co., Wis.	Gogebic	A_2	14	109	24	22	2	2.71	23	1.68	100	99	90	54	35	13	7	50
Jerome Co., Idaho	Portneuf	B_2	16	107	26	22	4	2.72	19	1.73	100	100	99	97	51	22	13	64
New Castle Co., Del.	Manor	A	17	109	35	26	9	2.71	23	1.61	100	89	77	50	42	30	16	51
DeSoto Co., Miss.	Grenada	C_2	18	106	41	22	19	2.70	19	1.73	100	100	109	100	67	30	23	70
Strafford Co., N.H.	Suffield	C_2	20	107	40	22	18	2.74	21	1.72	100	100	99	98	77	51	32	76
Maricopa Co., Ariz.	Mohave	B_2	20	109	43	20	23	2.66	11	1.96	100	99	91	73	50	40	28	64
Elbert Co., Ga.	Cecil	B_2	22	100	68	34	34	2.74	26	1.53	100	97	80	68	64	55	42	68
Madison Co., Iowa	Winterset	B_2	25	94	70	30	40	2.73	9	2.02	100	100	99	98	80	52	42	79
Albemarle Co., Va.	Davidson	B_2	31	89	72	37	35	2.89	24	1.69	100	100	99	95	89	82	75	90

[1] Sampled and named by the Soil Conservation Service, U.S. Department of Agriculture.
[2] Gradation percentages are based on the fraction passing the No. 4 sieve, the same fraction used in the compaction test.
[3] AASHO Road Test embankment; test data are average values of a cooperative materials testing program reported by Shook and Fang (*12*).

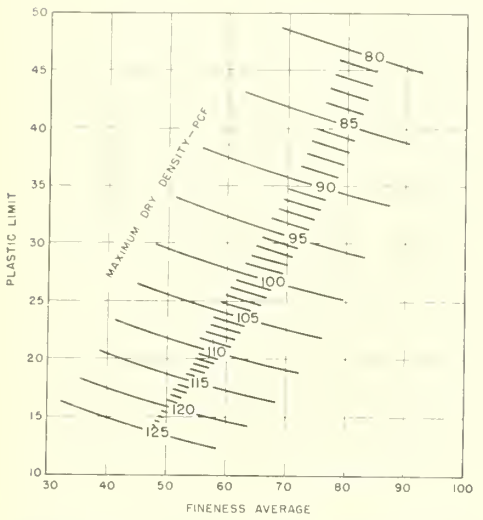

Figure 11.—Relation of maximum dry density to plastic limit and fineness average, analysis No. 8.

Table 7.—Comparison of methods for predicting optimum moisture content

Soil sample [1]	Optimum moisture content						
	Test [2]	Predicted by—					
		PL and FA	PL and LL	Jumikis	Turnbull	Davidson and Gardiner	Rowan and Graham
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
AASHO embankment	14	13	14	13	19	12	9
Gogebic	14	15	15	14	13	18	18
Portneuf	16	17	15	15	18	14	14
Manor	17	18	18	[3] 21	14	14	15
Grenada	18	18	18	22	20	21	16
Suffield	20	19	18	21	24	23	18
Mohave	20	16	18	23	19	14	7
Cecil	22	25	26	([4])	22	28	18
Winterset	25	25	24	([4])	28	18	6
Davidson	31	32	27	([4])	38	32	21
Sum of the deviations	0	13	16	([5])	28	37	63

[1] Description and basic data are given in table 6.
[2] Determined by AASHO Designation: T 99-57, Method A.
[3] Extrapolated.
[4] Beyond limits of predicting chart, figure 1.
[5] Insufficient data.

Table 8.—Comparison of methods for predicting maximum dry density

Soil sample [1]	Maximum dry density				
	Test [2]	Predicted by—			
		PL and FA	PL and LL	Davidson and Gardiner	Rowan and Graham
	P.c.f.	*P.c.f.*	*P.c.f.*	*P.c.f.*	*P.c.f.*
AASHO embankment	119	118	114	118	125
Gogebic	109	110	112	112	109
Portneuf	107	107	112	110	109
Manor	109	104	106	108	111
Grenada	106	106	106	99	105
Suffield	107	105	106	99	108
Mohave	109	110	106	111	125
Cecil	100	92	92	85	105
Winterset	94	94	95	98	126
Davidson	89	87	90	81	100
Sum of the deviations	0	20	30	52	76

[1] Description and basic data given in table 6.
[2] Determined by AASHO Designation: T 99-57, Method A.

selected to show how closely a regression formula would fit actual test results when the soils were from a relatively small area, where a restricted range of soil formation processes might exist. The actual optimum moisture contents ranged from 12 to 34 percent. The standard error of estimate was ±1.13 percent moisture.

●In regression analyses Nos. 6 through 9, the relationship of maximum dry density to several independent variables was investigated. In analysis No. 6, three independent variables that had appeared to be most closely related to maximum density during the examination of simple relationships were involved; these three were liquid limit, plastic limit, and fineness average. The data were used directly in the analysis without adjustment. The standard error of estimate was ±4.44 p.c.f.

●In analysis No. 7, maximum dry density was related to all of the independent variables in the sixth analysis and to optimum moisture content, which was known from the first Public Roads study to have a very good correlation with maximum dry density. All of the data were logarithmically transformed and adjusted where necessary. The standard error of estimate for this analysis was ±2.52 p.c.f.

●In analysis No. 8, maximum dry density was related to plastic limit and to fineness average, the number of variables being reduced to simplify the predicting equation. Although the number of variables was reduced from that used in analysis No. 6, the accuracy was slightly improved, probably because of the logarithmic transformations. The standard error of estimate was ±4.32 p.c.f. The predicting formula developed in analysis No. was:

$$\log \text{M.D.D.} = 7.247 - 0.567 \log (\text{PL}+20) - 0.110 \log \text{FA} \ldots (6$$

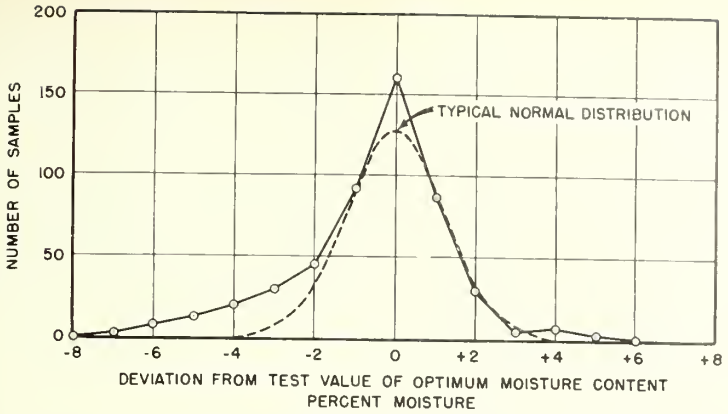

Figure 12.—Distribution of deviations of predicted optimum moisture contents, which was determined by the LL and PL method, from test results.

In figure 11, developed from this formula, maximum dry densities are shown for the range of plastic limits and fineness averages studied.

●Analysis No. 9 was performed in the same manner as No. 8 except that the data were limited to the 40 samples from Loudon Co., Tenn., for which the maximum dry densities ranged from 83 to 119 p.c.f. The standard error of estimate was ±2.98 p.c.f., which was smaller than that of analysis No. 8 and reflected the reduction in number of soil varieties.

Regression analyses summarized

To summarize the results of the regression analyses, the standard errors of estimate listed in table 4 showed that the two formulas developed for Loudon Co., Tenn., in analyses Nos. 5 and 9, fit the data much better than the corresponding formulas developed in analyses Nos. 1, 2, 3, 4, 6, and 8 for several States. Analysis 7 included the test optimum moisture content as an independent variable and, therefore, should not be considered with the other eight analyses. The standard errors of estimate of the Loudon County formulas were 1.1 percent moisture and 3.0 p.c.f. for optimum moisture content and maximum dry density, respectively. To show the relative magnitude of these standard errors of estimate, they may be compared to the averages of the test results examined. The standard error of 1.1 percent moisture was 5.3 percent $\left(\frac{1.1}{20.94}\times 100=5.3\right)$ of the average optimum moisture content (20.94); the standard error of 3.0 p.c.f. density was 2.9 percent $\left(\frac{3.0}{102.3}\times 100=2.9\right)$ of the average maximum dry density (102.30).

The standard errors of estimate of the formulas developed in analyses Nos. 1, 2, 3, 4, 6, and 8 for all the samples were approximately 2 percent moisture and 4.4 p.c.f. density. In terms of the average optimum moisture content and maximum dry density, 18.75 percent and 105.28 p.c.f. respectively, these standard errors of estimates were 10.7 and 4.2 percent. It is possible that combinations of the independent variables different from those used in this study could result in better correlations; only those combinations given in table 4 were analyzed. The selections of variables were based mainly on the data shown in table 3 and on the partial correlation coefficients developed in the analyses.

Table 9.—Comparison of actual [1] optimum moisture contents and maximum dry densities with those estimated by figures 9 and 11, respectively, based on plastic limit and fineness average for soils from Alaska and outside the United States

Soil		Optimum moisture content			Maximum dry density		
Location	Kind	Actual	Estimated	Deviation	Actual	Estimated	Deviation
		Percent	*Percent*	*Percent*	*P.c.f.*	*P.c.f.*	*P.c.f.*
Alajuela, Costa Rica	Lateritic	41	32	−9	77	87	+10
Las Lomas, Panama	do	29	32	+3	88	85	−3
Liberia	do	20	21	+1	107	96	−11
Do	do	12	12	0	126	120	−6
Do	do	15	14	−1	117	112	−5
Addis Ababa, Ethiopia	Black Cotton	39	38	−1	80	80	0
Kodiak, Alaska	Silty alluvium	20	21	+1	103	98	−5
Do	do	24	25	+1	95	93	−2
Do	do	33	30	−3	80	84	+4
Kenai-Kasilof, Alaska	do	10	9	−1	131	130	−1
Do	do	14	15	+1	118	114	−4
Do	do	13	12	−1	124	120	−4
Sum of the deviations				23			55

[1] Determined by AASHO Designation: T 99-57, Method A

Comparison of Predicting Methods

Optimum moisture content

To test the formula from analysis No. 4 (see fig. 9) for predicting optimum moisture content from plastic limit and fineness average, data for 10 soils that varied considerably in characteristics and that were from widely

Table 10.—Simple correlation coefficients between pairs of variables used in analyses Nos. 1 and 3 for optimum moisture content

VARIABLES—ANALYSIS NO. 1

	O.M.C.	LL	PL	PI	D'_{50}	FA	0.001
O.M.C.	1.00						
LL	0.87	1.00					
PL	0.91	0.84	1.00				
PI	0.72	0.95	0.62	1.00			
D'_{50}	0.47	0.45	0.35	0.45	1.00		
FA	0.75	0.77	0.64	0.74	0.73	1.00	
0.001	0.76	0.90	0.66	0.91	0.50	0.84	1.00

VARIABLES—ANALYSIS NO. 3

	Log O.M.C.	Log LL	Log PL	Log (PI+15)	Log $\frac{(D'_{50})}{100}$	Log (FA+100)	Log (0.001 fract. +40)
Log O.M.C.	1.00						
Log LL	0.89	1.00					
Log PL	0.90	0.83	1.00				
Log (PI+15)	0.74	0.94	0.59	1.00			
Log $\frac{(D'_{50})}{100}$	0.76	0.82	0.63	0.81	1.00		
Log (FA+100)	0.77	0.77	0.63	0.74	0.92	1.00	
Log (0.001 fract.+40)	0.76	0.88	0.62	0.92	0.90	0.84	1.00

separated geographical areas were selected from the files. These data are given in table 6. The predicted and test optimum moisture contents are shown in table 7, as are the optimum moisture contents predicted by the first Public Roads study (PL and LL) and by methods developed by Jumikis (*2*), Turnbull (*6, 7*), Davidson and Gardiner (*5*), and Rowan and Graham (*3*). The results indicated that the PL and FA method is a slightly better predictor than the PL and LL method. The other four methods are at a disadvantage in this type of comparison because they were developed for a more limited range of soil types (origin). Some predictions by each method were quite accurate.

Maximum dry density

To test the formula from analysis No. 8 (see fig. 11) for predicting maximum dry density from plastic limit and fineness average, data for the 10 soils described in table 6 again were used. The actual and predicted maximum dry densities are shown in table 8. This summary also shows the densities predicted by use of the Public Roads method from the first study (PL and LL) and by methods proposed by Davidson and Gardiner (*5*) and by Rowan and Graham (*3*). The densities predicted on the basis of plastic limit and fineness average were generally closer to the actual maximum dry densities, although for a few soils one of the other methods provided a closer estimate.

Another test of the formulas from analyses 4 and 8 was made using data from Alaska and areas outside the continental United States. The comparisons of estimated and actual test optimum moisture contents and maximum dry densities are given in table 9. The soils used in this test were the same, except for the Hawaiian soils, as those used in the first Public Roads study to evaluate the PL and LL method (table 2). Estimates for the Hawaiian soils could not be made because the grain-size analyses were not available. The sums of the deviations shown in table 9 are 23 percent moisture and 55 p.c.f. density, and they represent about half of the corresponding sum of deviations resulting from the estimates made with the methods developed in the first study.

Supplemental Information

To determine whether the deviations of the predicted optimum moisture contents from the actual optimum moisture contents have a normal distribution, the deviations listed in table 1 were plotted in figure 12. The resultant curve closely approximates a standard normal distribution curve; this is evidence that the standard error of estimate is a reasonable measure of the accuracy of the predicting methods. Simple correlation coefficients between pairs of variables used in analyses Nos. 1, 3, 6, and 7 are given in tables 10 and 11; the larger the coefficient, the better the correlation, 1.00 being perfect.

Table 11.—Simple correlation coefficients between pairs of variables used in analyses Nos. 6 and 7 for maximum dry density

	VARIABLES—ANALYSIS NO. 6			
	M.D.D.	LL	PL	FA
M.D.D.	1.00			
LL	0.81	1.00		
PL	0.89	0.84	1.00	
FA	0.70	0.77	0.64	1.00

	VARIABLES—ANALYSIS NO. 7				
	Log M.D.D.	Log (O.M.C.+15)	Log LL	Log (PL+20)	Log FA
Log M.D.D.	1.00				
Log (O.M.C.+15)	0.97	1.00			
Log LL	0.81	0.89	1.00		
Log (PL+20)	0.90	0.90	0.83	1.00	
Log FA	0.67	0.74	0.75	0.61	1.00

Tabulation Available

The test data used in the multiple linear regression analyses have been tabulated and are available to other researchers who would like to make additional studies of the data. This eight-page tabulation is available from the Chief, Physical Research Division, Bureau of Public Roads, U.S. Department of Commerce, Washington 25, D.C. In addition to the basic classification and compaction test data for 527 soils, the tabulation lists the location from which each soil was sampled, the soil series name, the soil type or textural classification of the "A" horizon, and the horizon actually sampled.

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APPENDIX I

Computer Curves for the PL and LL Analyses

Two additional analyses have been made to check the chart, figure 4, developed in the first Public Roads study. The analyses provided the following equations:

M.D.D. = 139.233 − 0.1344 LL − 1.185 PL

O.M.C. = 0.8151 + 0.1358 LL − 0.5290 PL

The standard errors of estimate were 4.69 p.c.f. for maximum dry density and 2.15 percent for optimum moisture content.

The charts developed from the two equations are given in figures 13 and 14. The curves were drawn within the range of the data examined. A comparison of these new curves with those given in figure 4 shows that the new curves generally have the same slope and spacing as the left-hand, straight-line portions of the old curves and that, for soils with maximum dry densities greater than 100 p.c.f., the curves almost coincide.

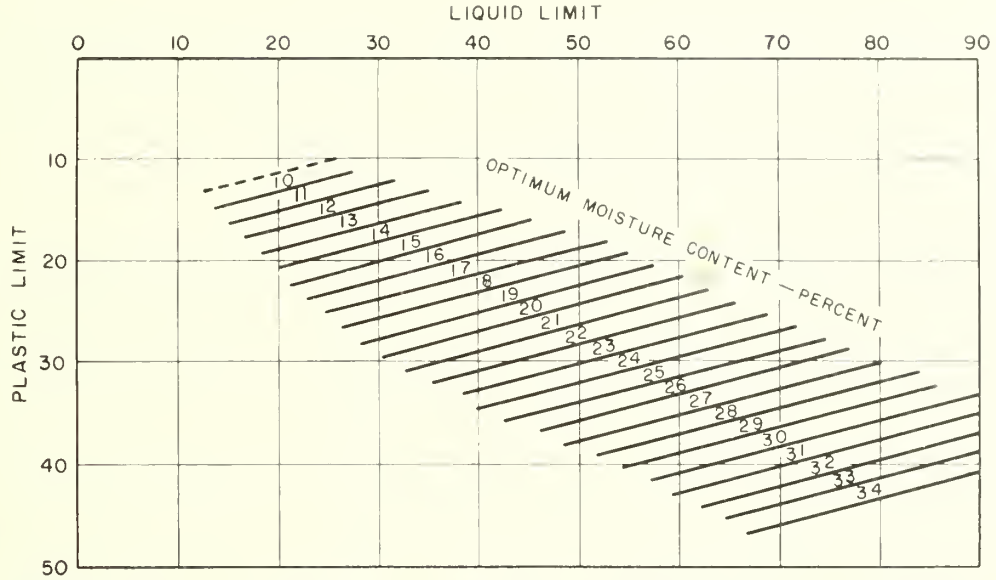

Figure 13.—Relation of optimum moisture content to plastic limit and liquid limit.

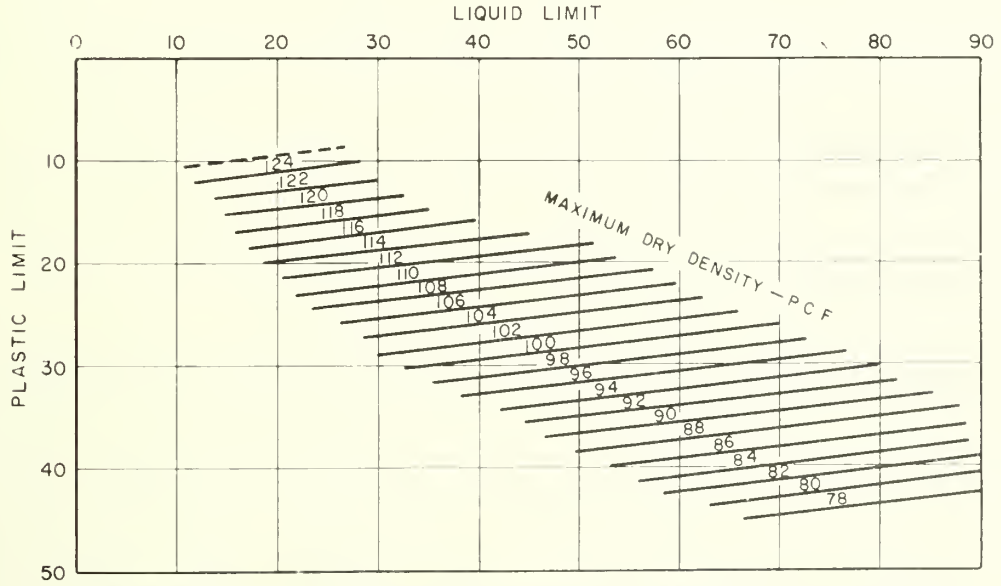

Figure 14.—Relation of maximum dry density to plastic limit and liquid limit.

Comparison of Properties of Coal-Modified Tar Binder, Tar, and Asphalt Cement

BY THE PHYSICAL RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

Reported[1] by WOODROW J. HALSTEAD, Supervisory Chemist,
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This article presents information on a study conducted to determine whether a newly developed product, coal-modified tar binder, could be used in the same manner as asphalt cement for a binder in pavement construction. Results of the laboratory study of the properties of this binder, a material produced by the high-temperature digestion of finely divided coal in tar and high boiling tar oils, are given. Also discussed is the comparison made of the coal-modified binder's properties and those of an ordinary tar pitch, RT-12 tar, and 85-100 penetration grade asphalt cement. Properties of laboratory mixtures containing each of the three binders also were compared.

This study indicated that coal-digestion increased the tar's resistance to hardening and apparently made it somewhat less brittle than an ordinary tar pitch. The improvements in these properties were not considered sufficient to warrant considering the coal-modified tar binder as a substitute for asphalt cement. Immersion-compression tests indicated that mixtures made with the coal-modified tar binder had greater resistance to deterioration from water than similar mixtures made with asphalt cement.

On the basis of this work, it is suggested that the coal-modified tar binder be considered as an improved tar and that it be used under the same conditions and for the same general purposes as unmodified tars.

Introduction

CONSIDERABLE interest was aroused when the Curtiss-Wright Corp. announced the development of a new coal-based road binder as the result of a research program that had been conducted in an effort to find new uses for coal and coal products. The great interest in this material by groups in several States, especially those seeking ways to use more coal in distressed, coal-producing areas, and the potential effect of such a binder on the national highway program prompted the Bureau of Public Roads to consider this development carefully. This article summarizes the results of the tests made and the findings of Public Roads concerning this material.

The basic principle used in the preparation of the new binder is the simultaneous digestion of powdered coal in coal tar and tar oils at a temperature of 500° to 600° F. It was claimed that, by adjusting the proportions of tar, tar oils, and coal, binders could be prepared covering the same penetration range as asphalt cements. It was the intent of the developer that the modified binders would be used in the same manner as asphalt cements in hot plant mixtures.

The digestion of powdered coal in tars and pitch has been used for a number of years in pipeline coatings and in pitches for steep, built-up roofs. No previous attempt has been made in this country to use this principle in the manufacture of a binder for pavements, but studies of the effects of powdered coal on the properties of road tar have been conducted recently in South Africa (*1*).[2] The details of the manufacture of the coal-modified tar binder and background information concerning its development are discussed in a report issued Dec. 30, 1960, by the Curtiss-Wright Corp., Research Division (*2*), and the description of the construction and performance of the first experimental pavements built with the new binder is given in two reports by the Kentucky Department of Highways (*3, 4*).

Through the courtesy of the Curtiss-Wright Corp. and the Kentucky Department of Highways, samples of the coal-modified tar binder and various materials were obtained for this study. Laboratory studies were conducted to determine the physical properties of the new binder and to compare these properties with the properties of ordinary tar and of asphalt cement. Two series of tests were made: In the first, a sample of the coal-modified tar binder obtained directly from the Curtiss-Wright Corp. was compared with a waterproofing tar pitch. In the second, tests were made to compare the coal-modified tar binder, the base tar, RT-12, and the asphalt cement used in one of the experimental pavement sections constructed in Kentucky.

Summary and Conclusions

These comparative tests showed that the coal digestion and the addition of high boiling tar oils, employed in the manufacture of the coal-modified tar binder, reduced the viscosity-temperature susceptibility of the tar and made it somewhat more resistant to hardening at high temperatures. However, comparison of this binder with asphalt of a similar softening point (85–100 penetration grade) showed that the coal-modified tar binder retained characteristics more nearly equal to those of the tar than those of the asphalt. In particular, the volatile loss and hardening in heat tests of the two tar products were about the same, but these heat tests results were significantly different from the results of similar tests on asphalts. At all test temperatures, stabilities as indicated by compressive strengths, for mixtures made with the coal-modified tar

[1] Presented at the 41st annual meeting, Highway Research Board, Washington, D.C., January 1962.

[2] References indicated by italic numbers in parentheses are listed on page 95.

binder were higher than stabilities of similar mixtures made with asphalt. However, the stabilities of the coal-modified tar binder obtained at low temperatures were such that a lack of flexibility and possible brittleness might be suspected. The subsequent behavior of the materials in the Kentucky experiments verified this brittleness and lack of flexibility.

The results of this study, in general, showed that coal-modified tar binders, such as the Curtiss-Wright material, should be considered as an improved tar. It would be expected to perform better than unmodified tars in a number of applications. However, it also was indicated that the precautions normally employed when using tars should be employed when using this material and that an attempt should not be made to substitute coal-modified tar binder for penetration grade asphalts.

Comparison of a Modified Tar Binder with a Tar Pitch

In the first series of tests, the properties of the coal-modified tar binder were compared to the properties of a tar pitch meeting an AASHO specification, *Standard Specification for Coal-Tar Pitch for Roofing, Dampproofing, and Waterproofing*, AASHO Designation: M-118, Type B. Table 1 gives the results of these tests. The tar pitch used in these comparisons was selected so as to have the same softening point as the modified binder. The penetration at 77° F. also was about the same for both materials. However, the absolute viscosity results, determined at different temperatures by the Koppers vacuum-operated capillary tube viscometer (*5*) showed the coal-modified tar binder to have a somewhat lower viscosity-temperature susceptibility. The viscosity-temperature coefficient, as indicated by the slope of the line obtained when log log absolute viscosity in centipoises was plotted against log absolute temperature in degrees Rankine (° F. plus 459.7), was −4.71 for the coal-modified tar binder and −4.99 for the tar pitch. Penetration tests made at different temperatures also indicated lower susceptibility for the coal-modified tar binder, the slope of the log penetration versus temperature curves being 0.0258 and 0.0427 for the coal-modified tar binder and the tar pitch, respectively.

In the thin-film oven test (⅛-in., five hours) run at the standard temperature of 325° F. (*Standard Method for Thin-Film Oven Test*, AASHO Designation: T 179–60), both materials had very high losses and exhibited extreme hardening. As this temperature is higher than one that likely would be used for materials of this nature, tests were repeated at a temperature of 250° F. At this lower temperature, the coal-modified tar binder showed a greater resistance to hardening than did the tar pitch. Impact resistance, as measured by the height at which a one pound steel ball produced fracture of a rigidly supported half-inch cube of material at the indicated test temperature, was greater for the modified binder than for the tar pitch. Except for ductility, the results of tests made on the residues from the thin-film oven tests at 250° F. showed the same general trends as the results of tests on the original materials. No specific conclusions regarding comparable ductility can be drawn. The original ductility of the modified binder was higher at 77° F. than that of the tar pitch, but at 60° F. the reverse was true. At 77° F., the 250° F. residue of the tar pitch was more ductile than the coal-modified tar binder even though it was harder, as judged by the penetration test. At 60° F., the tar pitch had no ductility compared to a ductility of 8 centimeters for the coal-modified tar binder.

This series of tests confirmed the claim that the coal-modified tar binder has properties that are an improvement over those for tar pitch; they also indicated that further research was warranted to compare the properties of the new binder with asphalt cement and to determine the relative behavior of these two materials when used in paving mixtures.

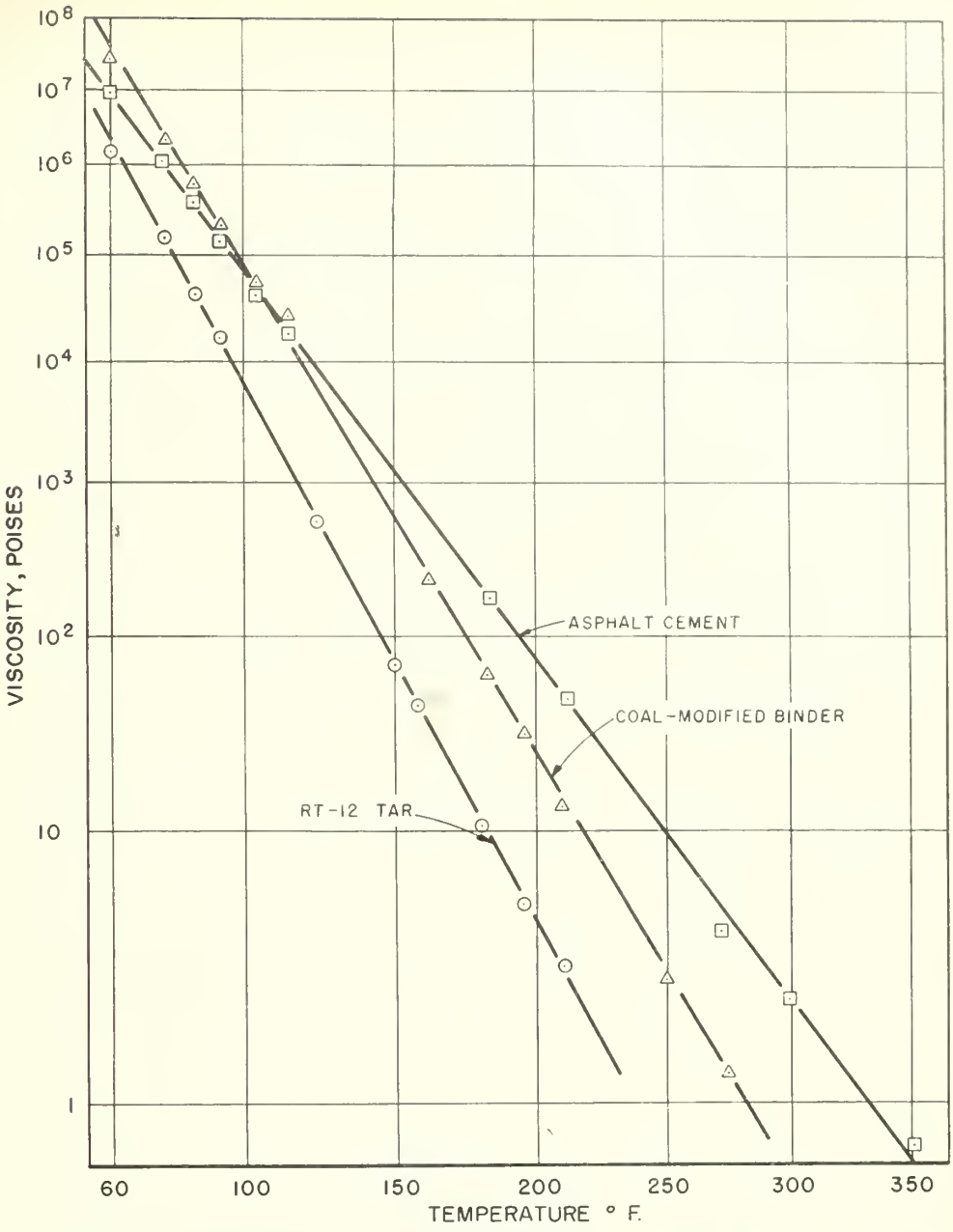

Figure 1.—Viscosity-temperature relationships for binders used in Kentucky experimental project.

Comparison of Properties of Binders

The second and more extensive series of tests was made on samples of materials used in the Kentucky experimental paving project 11 (urban) located on U.S. 25 in London, Ky. The materials included the coal-modified tar binder, the RT-12 tar used as the base for the binder, the 85–100 penetration grade asphalt cement used in the control sections, the powdered coal, and the aggregate used in the mixtures. The properties of the three

Table 1.—Comparison of properties of coal-modified binder with tar pitch

		Coal-modified tar binder	Waterproofing pitch
Softening point (R&B)	° F	117	117
Specific gravity at 77° F		1.248	1.269
Solubility in CS_2	percent	74.6	81.1
Ductility at 77° F. (AASHO T 52)	do	2.4	6.7
Absolute viscosity, poises, at:			
140° F		1335	747
122° F		594	310
158° F			160
167° F		130	80.1
173.5° F		92.7	
176° F			44.0
Viscosity-temperature susceptibility [1]		−4.71	−4.99
Penetration at:			
77° F., 100 g., 5 sec		63	62
53.4° F., 100 g., 5 sec.		15	6
39.2° F., 200 g., 60 sec		23	11
Penetration-temperature susceptibility [2]		0.0258	0.0427
Ductility, 5 cm. min., cm., at:			
77° F		51	41
60° F		30	59
Impact resistance,[3] inches to fracture, at:			
77° F		9+	5
60° F		5	2
53.5° F		5	2
Thin-film tests at 325° F. ($\frac{1}{8}$-in. film, 5 hrs.):			
Loss	percent	11.4	9.4
Softening point of residue	° F	170	162
Penetration of residue, 100 g., 5 sec. at 77° F.		0	0
Thin-film tests at 250° F. ($\frac{1}{8}$-in. film, 5 hrs.):			
Loss	percent	1.4	2.9
Penetration of residue, at:			
77° F., 100 g., 5 sec		27	15
53.4° F., 100 g., 5 sec.		5	1
39.2° F., 200 g., 60 sec.		8	1
Retained penetration at 77° F.	percent	43	24
Penetration-temperature susceptibility [2]		0.0310	0.0498
Ductility, 5 cm./min., cm., at:			
77° F		38	61
60° F		8	0
Impact resistance,[3] inches to fracture, at:			
60° F		2	$\frac{3}{4}$
53.4° F		$1\frac{1}{4}$	1

[1] Viscosity-temperature susceptibility $= \frac{\log \log v_2 - \log \log v_1}{\log T_2 - \log T_1}$

v = viscosity in centipoises.
T = Temperature in degrees Rankine (° F. + 459.7).

[2] Penetration-temperature susceptibility $= \frac{\log P_2 - \log P_1}{t_2 - t_1}$

P_1 = Penetration at temperature t_1.
P_2 = Penetration at temperature t_2.
t = temperature, degrees F.

[3] One pound steel ball on $\frac{1}{2}$-inch cube.

binders were determined to show their relative viscosity-temperature susceptibilities and their degree of hardening when heated. Comparisons were made of the properties of laboratory mixtures prepared with each binder and the aggregate used in the Kentucky experiment. Comparisons also were made for mixtures of each of the three binders and other selected aggregates.

Table 2 gives the results of the tests made on the binders. These include normally determined physical characteristics and absolute viscosities, in poises, at various temperatures ranging from about 60° F. to 350° F. To determine viscosities over this wide range of temperatures, three instruments were used: the Shell sliding plate microviscometer (*6*) at the lower temperature range of 60° F. to 115° F.; the Koppers vacuum-capillary viscometer in the intermediate temperature range of 115° F. to 200° F.; and the Saybolt furol viscometer, with conversions being made to absolute viscosity in the higher temperature range of 200° F. to 350° F.

Figure 1 shows the viscosity-temperature relations of the three binders used in the Kentucky project. Although viscosity in poises and temperature in degrees Fahrenheit are indicated in the figure, these curves were obtained by plotting log log viscosity in centipoises against log of absolute temperature in degrees Rankine. A good, straightline relation is indicated by the data points for all the binders even though the viscosities were obtained by the different instruments. The difference in slope of the curves for the asphalt and the coal-modified tar binder is significant. The RT–12 tar had substantially lower viscosities for the same temperatures, but it was only slightly more susceptible to temperature than the coal-modified material. The calculated slopes of these lines are: −4.78, −4.38, and −3.51 for the tar, coal-modified tar binder, and the asphalt cement, respectively.

Figure 2 shows the furol viscosities at various temperatures for the three binders. These curves were used to determine the various mixing temperatures for the studies of laboratory mixtures to be discussed later. It should be noted that furol viscosity here is directly related to kinematic viscosity in stokes rather than to absolute viscosity in poises. The data points shown in these curves are the basis fo the absolute viscosities, in poises, given i table 2 for the temperatures indicated.

Resistance to hardening

One of the claims of the developers, tha coal-modified tar binder could be used in th same manner as asphalt cement in bituminou concrete construction, was of much interes because it is well known that ordinary tars ar more volatile than asphalts and should not b subjected to the same high temperatures a asphalt during the production of hot pavin mixtures. Because of this claim, the relativ resistance of these two binders to hardenin when subjected to heat was of considerabl interest. To compare this heat resistance, th thin-film oven test ($\frac{1}{8}$-in., 5 hours), which is commonly used as a specification test to evalu ate the hardening characteristics of asphal cements, was employed. Tests were made a temperatures ranging from 210° F. to 325° F on the three binder materials. In addition the oven-loss test (*Standard Method of Test fo Loss on Heating of Oil and Asphaltic Com pounds*, AASHO Designation: T 47–42) wa made on the coal-modified tar binder and th asphalt cement. The results of these tests ar given in table 3.

Figure 3 shows the comparison of the weigh loss during the thin-film tests of the asphal cement, the coal-modified tar binder, and th RT–12 tar at the various test temperatures Although this particular asphalt cemen showed small gains in weight, most asphal cements of this grade have small weight losse in this test. In the study of asphalt cement recently conducted by Public Roads (*7*), th highest loss shown by any of the 85–100 pene tration grade asphalts was 2.18 percent Eighty-seven percent of the 85–100 material in that investigation had losses of 0.5 percen or less. The coal-modified tar binder wa different from asphalts with respect to vola tility, as evidenced by its loss of 10 percent a 325° F. The loss for the RT–12 tar was 11. percent at 325° F.; thus, the results for th two tar materials generally were comparable

Figure 4 illustrates the relative hardenin characteristics of the three binders as measure by the percent of original penetration retaine after the thin-film test at various test tem peratures. The wide differences between th tar products and the asphalt again wa illustrated. Different asphalts, of course show different percentages of retained penetra tion, but the percentage never would be a low as that shown by the tar products. Fo example, the asphalt showing the highest los encountered with 85–100 penetration grad materials, which was described in the pre ceding paragraph, had a retained penetration of 33 percent. This is considerably highe than the 1 and 3 percent shown by the tw tar materials under the same conditions.

Note that, although the hardening o asphalt in the thin-film oven test has bee related to the hardening in an asphalt during mixing in a hot plant, no information i available concerning the relation of th hardening of tar materials in such tests to th actual hardening occurring in the mixing an laying operations. Because of the relatively

arge amount of volatile matter in tar products, is possible that the long time of oven xposure, compared to the time the mixture is t high temperature prior to compaction on he road, would produce considerably greater ardening in these materials than actually ould be encountered in construction opera- ons. Nevertheless, it is believed that the elationships shown are important: They mphasize the fact that the coal-modified tar inder has properties different from those of sphalt and, therefore, it is not realistic to onsider this material as an alternate to sphalt in all respects.

Studies of Laboratory Mixtures

To further compare the behavior of the oal-modified tar binder, RT–12 tar, and sphalt cement, the characteristics of labora- ory prepared mixtures made with each binder ere compared under several conditions. The hanges in stability after various periods of ven aging and after various periods of mmersion in water were determined. Another series of stability tests, at different emperatures, was made to evaluate the effect f the previously observed differences in the iscosity-temperature relations of the binders.

Because of the relatively wide differences in he specific gravities of the three binders, all f the laboratory mixtures prepared for the arious phases of this part of the study were esigned using equal volumes of binder per nit weight of aggregate (5.85 milliliters of inder per 100 grams of aggregate). This as considered to provide a better basis for omparison than test mixtures prepared with ercentages of binder on an equal weight asis. Because of the differences in tem- erature susceptibility, the mixing and mold- ng temperatures were controlled so that the urol viscosities of the binders were approxi- nately the same for each individual phase of he study.

The aggregates for all of the mixtures were eated overnight at the predetermined mixing emperatures. The binders were heated to his mixing temperature just prior to the mix- ng with the preheated aggregates in a modified obart mechanical mixer. The mixing period as 2 minutes. Marshall specimens were repared in a mechanical compactor with 50 blows of the hammer applied to each face of he specimen. The specimens for the uncon- fined compression test were molded by the louble-plunger method with a 3,000 p.s.i. load held for 2 minutes.

Marshall Stability

To determine the effect of laboratory aging tests on the strength of mixtures as measured by Marshall stability, specimens were prepared with the three binders, as received. In addition, specimens were prepared with the RT–12 tar in which 10 percent of the tar was replaced by powdered coal, in order to obtain some measure of the filler effect of the powdered coal. As stated previously, all of the mixtures were proportioned with the

Table 2.—Test properties of Kentucky binders

	RT-12 tar	Coal-modified tar binder	Asphalt cement
Specific gravity at 77°/77° F.	1.265	1.267	1.019
Penetration at 77° F., 100 g., 5 sec.	235	71	92
Softening point (R&B), °F.	96	115	116
Ductility 77° F., 5 cm./min., cm.		54	182
Solubility in CS_2, percent	79.5	71.3	[1] 99.9
Organic insoluble, do	20.5	28.0	0.1
Distillation test (AASHO T 52) to:			
455° F., percent	0.0	0.0	
518° F., do	0.3	0.3	
572° F., do	3.8	2.7	
Residue, do	95.7	97.2	
Softening point of residue, °F.	113	128	
Absolute viscosity, poises, at:			
61.5° F.	1,500,000	24,500,000	8,800,000
77° F.	160,000	2,300,000	1,200,000
86° F.	42,000	660,000	418,000
95° F.	17,500	200,000	169,000
105° F.		56,000	44,000
115° F.		28,000	20,000
123.6° F.	492		
149.4° F.	65.7		
157.9° F.	40.9		
160.3° F.		217	
180° F.	10.3	[2] 60.1	[2] 169
195° F.	4.92	[3] 29.0	[3] 43.2
210° F.	2.77	12.8	[4] 43.2
250° F.		2.52	
275° F.		1.26	4.35
300° F.			2.18
350° F.			.747
Viscosity-temperature susceptibility [5]	−4.78	−4.38	−3.51

[1] Solubility in carbon tetrachloride.
[2] Actual temperature, 180.8° F.
[3] Actual temperature, 195.9° F.
[4] Actual temperature, 211.7° F.
[5] $\frac{\log\log v_2 - \log\log v_1}{\log T_2 - \log T_1}$

v = viscosity in centipoises.
T = Absolute temperature in degrees Rankine (° F. + 459.7).

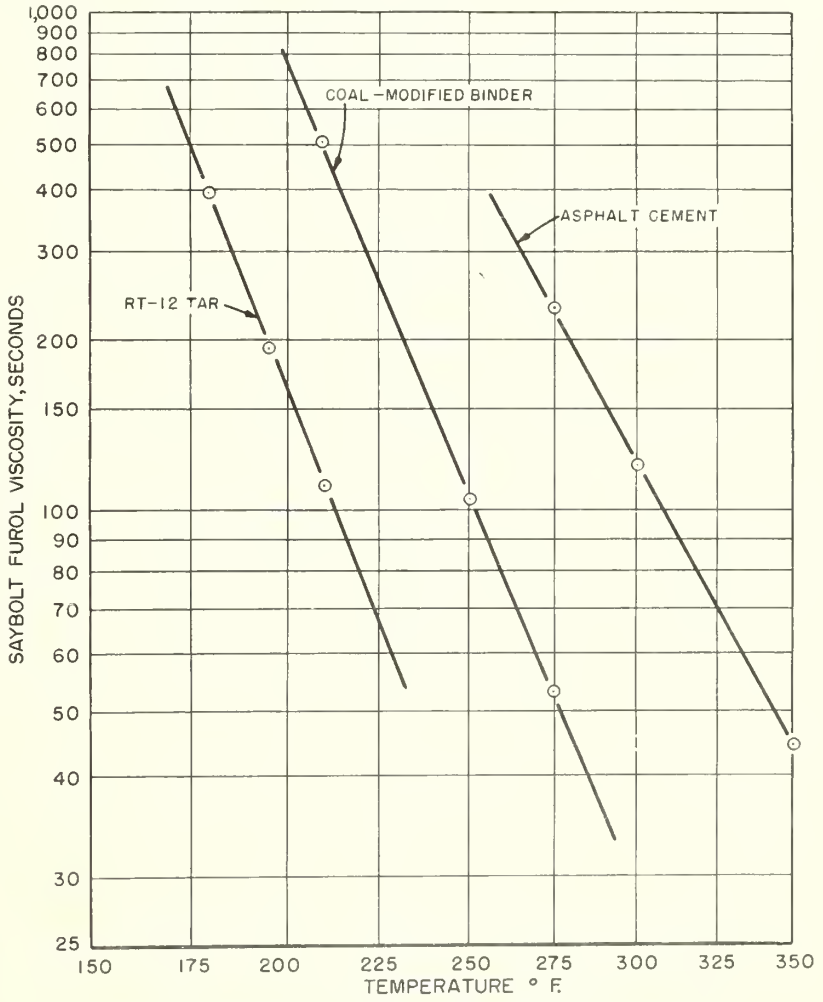

Figure 2.—Furol viscosity of binders at various temperatures.

Table 3.—Effect of oven exposure at various temperatures

	RT-12 tar	Coal-modified tar binder	Asphalt cement
Oven loss test (AASHO T-47):			
Loss ..... percent	[1] ----------	2.36	0.02
Penetration of residue at 77° F., 100 g., 5 sec	----------	40	80
Retained penetration ..... percent	----------	56	65
Thin film tests (⅛ in. film, 5 hrs.):			
At 210° F.:			
Loss ..... [1] percent	1.46	1.05	+0.08
Penetration of residue, at 77° F., 100 g., 5 sec	128	50	79
Retained penetration ..... percent	54	70	86
Ductility at 77° F., 5 cm. per min ..... cm	73	46	228
Softening point ..... degrees F.	103	121	119
At 250° F.:			
Loss ..... [1] percent	3.60	2.68	+0.23
Penetration of residue at 77° F., 100 g., 5 sec	71	31	75
Retained penetration ..... percent	30	44	82
Ductility at 77° F., 5 cm. per min ..... cm	151	42	239
Softening point ..... degrees F.	114	129	121
At 275° F.:			
Loss ..... [1] percent	5.67	4.47	+0.25
Penetration of residue at 77° F., 100 g., 5 sec	41	18	68
Retained penetration ..... percent	17	25	74
Ductility at 77° F., 5 cm. per min ..... cm	75	27	230
Softening point ..... degrees F.	122	137	122
At 325° F.:			
Loss ..... [1] percent	11.35	9.98	+0.11
Penetration of residue at 77° F., 100 g., 5 sec	3	2	60
Retained penetration ..... percent	1	3	65
Ductility at 77° F., 5 cm. per min	1	[2]	247
Softening point ..... degrees F.	150	164	126

[1] The + sign indicates gain in weight.
[2] Not run, specimen could not be prepared properly.

binder on an equal volume basis and were mixed and molded at an approximately equal furol viscosity. After molding, one group of specimens was tested immediately and the balance was aged in an oven at a temperature of 140° F. for periods ranging from 1 to 30 days. After each aging period, Marshall stability was determined at a temperature of 140° F. The stabilities obtained, along with pertinent information on proportioning and mixing, are given in table 4. The effect of aging, illustrated in figure 5, shows that the stability for all materials increased with aging in the oven, and that the increase in stability for the coal-modified tar binder was significantly greater than that obtained with either the asphalt or the RT–12 tar.

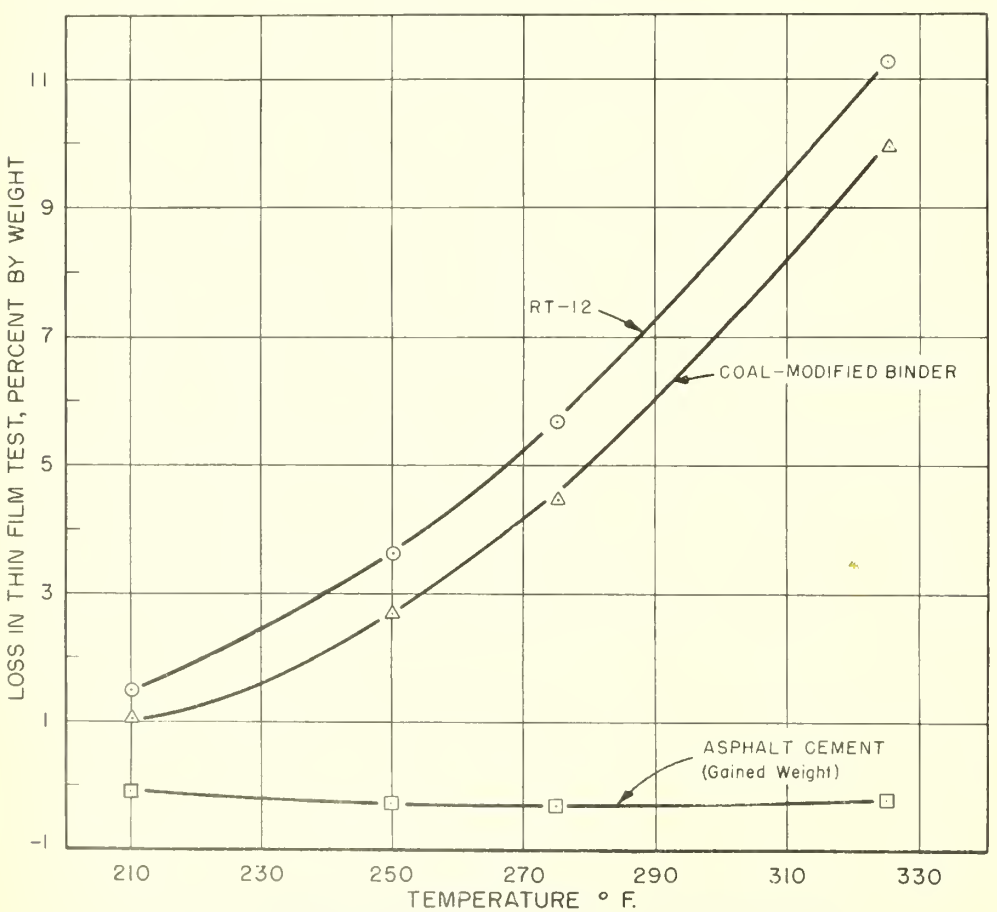

Figure 3.—Weight loss in thin-film test at various temperatures.

The previous oven-loss tests had indicated that the RT–12 tar had slightly greater volatility than the coal-modified tar binder and, therefore, a somewhat greater increase in stability would be expected for the RT–12 mixture if such an increase were attributable to volatility alone. Also, if the effect of the powered coal were primarily a mechanical effect of increased fine material, the stabilities of the RT–12 tar mixtures to which the powdered coal had been added would be expected to more nearly equal those obtained with the mixtures of coal-modified tar binder than the stabilities of the RT–12 mixtures. However, as can be seen, the data did not support such suppositions. Although the mixtures containing the powdered coal did have higher stabilities for the same conditions, this increase in stability accounted for only a very small proportion of the total increase in stability shown by the coal-modified tar binder mixtures. These results indicate that some factor, such as the development of internal structure, contributes to a considerably greater increase in the stability of the coal-modified tar binder than can be accounted for by the increase in viscosity caused by loss of volatile matter or by the effect of coal as a filler.

Effect of aging

To further explore the relative tendency of the three binders to exhibit a difference in characteristics upon aging, a special series of viscosity tests was conducted, as follows:

Sliding plate viscosity specimens were prepared in the usual manner for viscosity determination with the microviscometer, and the initial viscosity was determined at 77° F. within 15 minutes after preparation of the specimens had been completed. The sides of the glass plates then were taped with a nonpermeable plastic tape to minimize volatile losses, and the specimens were placed in an oven at a temperature of 110° F. Absolute viscosity determinations at 77° F. were made on the same specimen after periods of 1, 5, 8, and 13 days. Immediately after the viscosity determination on the 13th day, the specimens were heated above their softening points to a temperature of 150° F. and the viscosity at 77° F. was redetermined. This procedure was used to obtain a measure of the nonpermanent hardening that had occurred. Results of these tests are given in table 5 and are shown graphically in figure 6. The vertical components of the lines in the figures at the 13-day period indicate the change in viscosity that occurred after the specimens were heated.

Although there are some inconsistencies in the data, viscosity of all three binders increased during oven aging. For the RT–12 tar, the viscosity increased when the specimen was heated above its softening point after 13

ays of aging. Thus any possible develop-ment of structure or reversible hardening was masked by a permanent hardening, which robably was caused by an unavoidable loss f volatile matter. However, for both the sphalt cement and the coal-modified tar inder, viscosity at 77° F. decreased signifi-antly after heating of the specimens to a emperature above the softening point and ecooling. This decrease amounted to approxi-nately 64 percent of the total increase for the oal-modified tar binder and to approximately 2 percent of the increase for the asphalt ement.

The tendency for bituminous materials to xhibit such hardening through the develop-ment of internal structure with time is well-ecognized. Lee and Dickinson (*8*) attributed his phenomenon in tars to the possible crystal-zation of constituents or to a change in degree f the dispersion of colloidal constituents. Brown, Sparks, and Smith (*9*) in discussing his phenomenon in asphalts attributed it to ". . . internal physical reorientation and re-rganization at the atomic, molecular, and nicelle levels, of the components of the as-halt." Although this study was not conclu-ive, the results of oven-aging tests on both he mixtures and binders indicate that the oal-modified tar binder may exhibit the tend-ney to develop structural hardening to a nuch greater extent than either an ordinary ar or an asphalt. The effect of this phenom-non on the behavior of the materials in pave-nents should be considered in any further esearch on such materials.

Effect of Water on Compressive Strength

An important property of a bituminous nix is its ability to resist loss of strength in he presence of water. To show this charac-eristic, immersion-compression tests were nade using two types of aggregate, a quartz-te and a granite, both of which were rela-ively sensitive to water, as had been shown by previous Public Roads studies. Each of he aggregates was crushed and then recom-bined to the same grading as that of the aggre-gate obtained from the experimental project n London, Ky. Only the coal-modified tar and asphalt binders were used in this phase of the study.

Table 6 gives the results of compressive strength tests on the specimens before im-mersion and after immersion in water at 120° F. for periods up to 18 days. The retained strengths and percentages of swell after each period also are given along with information concerning the mixture composition and mixing temperatures.

The results of the compressive strength test at 77° F. and the percent of retained strength after various periods of immersion are plotted in figure 7. These results show that the modi-fied tar binder had higher initial strengths than the asphalt cement with each aggregate. The percentages of retained strengths were also higher for specimens of each of the mixtures containing the coal-modified tar binder than for the comparable specimens containing

Table 4.—Effect of oven aging at 140° F. on Marshall stability

	RT-12 tar	RT-12 + coal [1]	Coal modified tar binder	Asphalt cement
Specific gravity of binder at 77°/77° F	1.265 [2]		1.267	1.019
Weight of binder per 100 g. of aggregate—g	7.40		7.41	5.96
Volume of binder per 100 g. of aggregate—ml	5.85		5.85	5.85
Mixing temperature [2]—° F	210	240	243	300
Marshall stability at 140° F. after aging in oven at 140° F. for: [3]				
0 days—pounds	132	139	321	238
1 days—do	180	240	532	292
3 days—do	243	304	817	336
10 days—do	311	456	1,334	410
30 days—do	571	856	2,569	574

[1] Ten percent by weight of the tar was replaced by powdered coal used in manufacturing coal-modified binder.
[2] Mixing temperature was adjusted to give approximate equal viscosity for all binders (120 Saybolt furol seconds).
[3] Average of 4 specimens. Aggregate was same as that used in Kentucky experiment and was graded, as follows:

Sieve size:	½ in.	⅜ in.	No. 4	No. 10	No. 20	No. 40	No. 80	No. 200
Percent passing:	100	98	72	47	34	17	3	1.4

Table 5.—Increase in viscosity with time of oven aging at 110° F.

Aging period [1]	Viscosity at 77° F.		
	RT-12 tar	Coal-modified tar binder	Asphalt cement
Days	*Poises*	*Poises*	*Poises*
0	163×10³	1,890×10³	1,230×10³
1	380	2,690	1,520
5	392	2,830	1,870
8	609	4,220	1,840
13	921	6,770	2,480
[2] 13–X	1,050	3,660	2,210

[1] Sealed specimens aged in oven at 110° F.
[2] Specimens heated to 150° F. after 13-day determination and viscosity determination repeated.

Table 6.—Effect of immersion in water at 120° F. on compressive strength

	Quartzite [1]		Granite [1]	
	Coal-modified tar binder	Asphalt cement	Coal-modified tar binder	Asphalt cement
Weight of binder per 100 g. aggregate—g	7.41	5.96	7.41	5.96
Volume of binder per 100 g. aggregate—ml	5.85	5.85	5.85	5.85
Mixing temperature [2]—° F	240	300	240	300
Compressive strength before immersion [3]—p.s.i.	258	151	363	257
Results [3] after 4 days of immersion:				
Compressive strength—p.s.i.	223	100	237	163
Retained strength—percent	86	66	65	63
Swell—do	0.1	0.5	1.0	1.0
Results [3] after 11 days of immersion:				
Compressive strength—p.s.i.	239	93	212	131
Retained strength—percent	93	62	58	51
Swell—do	0.2	0.8	1.3	1.5
Results [3] after 18 days of immersion:				
Compressive strength—p.s.i.	237	84	209	129
Retained strength—percent	92	56	58	50
Swell—do	0.2	1.4	1.4	1.5

[1] Gradation of both mixtures was as follows:

Sieve size:	½ in.	⅜ in.	No. 4	No. 10	No. 40	No. 80	No. 200
Percent passing:	100	98	72	47	17	3	1.4

[2] Mixing temperature was adjusted to give approximate equal binder viscosity (120–150 Saybolt furol seconds).
[3] All results based on average for three test cylinders, 4 inches in diameter and 4 inches high, tested at 77° F.

Table 7.—Variation of compressive strength with test temperature

	RT-12 tar	Coal-modified tar binder	Asphalt cement
Specific gravity of binder at 77°/77° F	1.265	1.267	1.019
Weight of binder per 100 g. aggregate—g	7.40	7.41	5.96
Volume of binder per 100 g. aggregate—ml	5.85	5.85	5.85
Mixing temperature [1]—° F	220	265	320
Compressive strength,[2] p.s.i., at:			
0° F	5,197	5,446	2,568
36° F	1,503	2,358	912
75° F	220	544	291
100° F	86	236	143

[1] Mixing temperature was adjusted to give approximately equal viscosities for all binders (70–100 sec.).
[2] Each result represents the average for three tests on cylinders, 3 inches in diameter and 3 inches high. Aggregate was composed of 58 percent crushed granite (½ in. to No. 10); 37 percent river sand (passing No. 10); and 5 percent lime-stone dust. Gradation was, as follows:

Sieve size:	½ in.	⅜ in.	No. 4	No. 10	No. 20	No. 40	No. 80	No. 200
Percent passing:	100	90	57	42	23	20	7	5.2

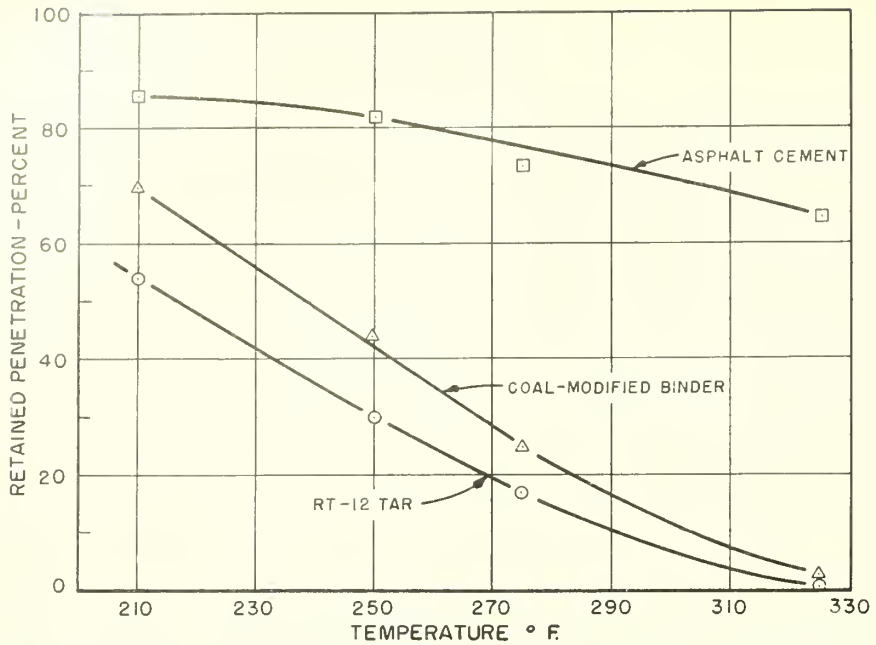

Figure 4.—Resistance to hardening at various test temperatures (⅛-in. film, 5 hours).

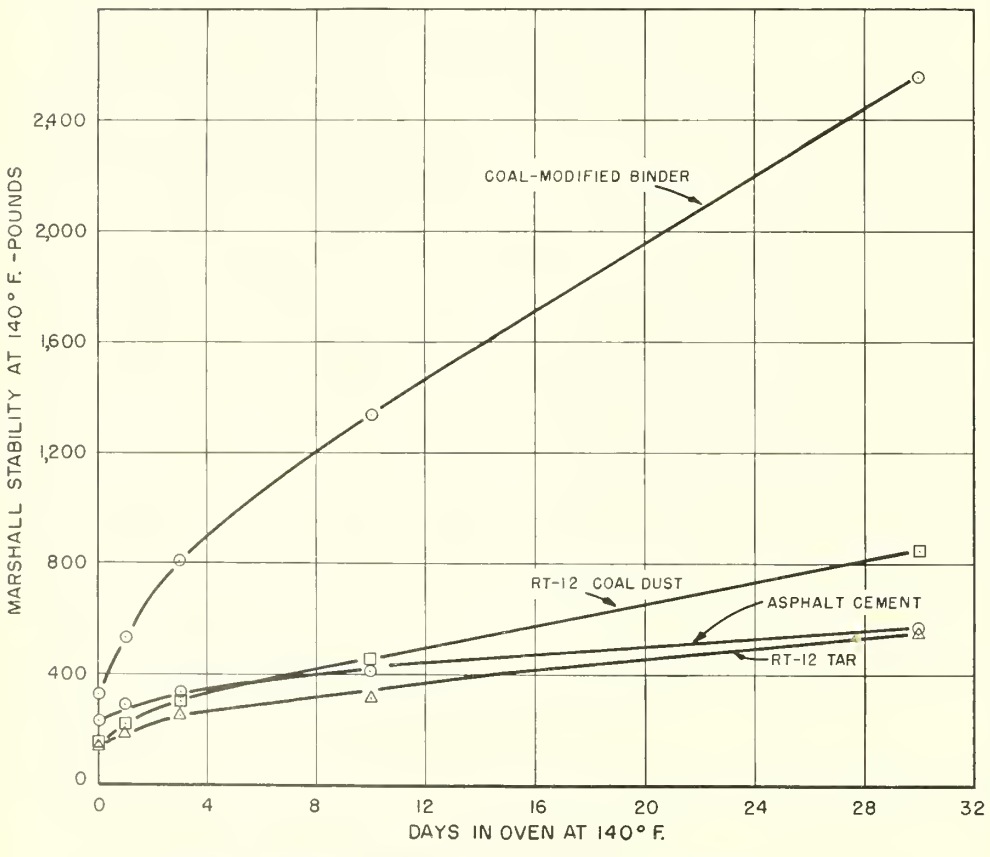

Figure 5.—Increase in Marshall stability during time of aging in oven.

asphalt and the same aggregate. Although the results for the mixtures made with quartzite aggregate and coal-modified tar binder were somewhat erratic, the strengths at 4 days being less than those at 12 and 18 days, these mixtures appeared to have been affected very little by water.

Effect of Temperature on Unconfined Compressive Strength

Because of the differences in viscosity-temperature susceptibility of the coal-modified tar binder, RT–12 tar, and asphalt binders the relative change in stability with temperature was of interest. For this series of tests compressive strength cylinders, 3 inches in diameter and 3 inches high, were used. As with the other test series, the binders were added on an equal volume basis and the mixtures were prepared at equal furol viscosities. The aggregate for this series of tests consisted of crushed granite as the coarse aggregate, river sand as the fine aggregate, and limestone dust as the mineral filler. The specimens were tested for stability by unconfined compressive strength tests over a range of temperatures from 0° F. to 100° F. The results of these tests, mixture composition, and mixing conditions are given in table 7.

The relationships of strength to temperature for the three binders are shown in figure 8. These results followed the generally expected pattern; they were in the same order as viscosity-temperature susceptibility of the binders themselves. Compressive strength decreased as the temperature increased, with the rate of decrease being greatest for the RT–12 tar and least for the asphalt cement. It is of interest to note that, when the curves for the asphalt and coal-modified tar binders were extended, the coal-modified tar binder showed higher strength at 140° F., the maximum temperature usually found in pavements. Thus it was indicated that the coal-modified tar binder of the consistency used in these tests would provide higher stabilities than the 85–100 penetration asphalts at any temperature that would be encountered in service. This was not true for the RT–12 tar; the curve intersects the asphalt curve at a temperature of 67° F., thus indicating that this material would have significantly lower stability at 140° F. It is of further interest to note the extremely high compressive strengths shown for the coal-modified tar and the RT–12 tar at 0° F. These strengths are approximately double those of the asphalt and are in the range of the strength of portland cement concrete.

The relation of the data obtained in these laboratory tests to stability of the pavement after construction is not known. However, the conditions of laboratory mixing, compaction, and testing were such that the hard-

ning occurring would most likely have been ss than actually occurs in construction. lso, the tar materials would be expected to xhibit greater differences between laboratory nd field specimens than asphalts. Thus, it an be surmised that actual differences in ne stabilities of pavements made with the ifferent materials would be greater than hose indicated by these tests.

As no evaluation of the brittleness or esistance to abrasion of the various mixtures as made, compressive strengths of the magitude indicated by the tar materials at low emperatures should not necessarily be conrued as being advantageous. It is likely hat pavements containing such mixtures ould be subject to abrasion losses and racking at low temperatures, and that such istress would be accelerated by any hardening of the binders in service. The performnce of these materials in the Kentucky xperiments seemed to indicate such deficncies.

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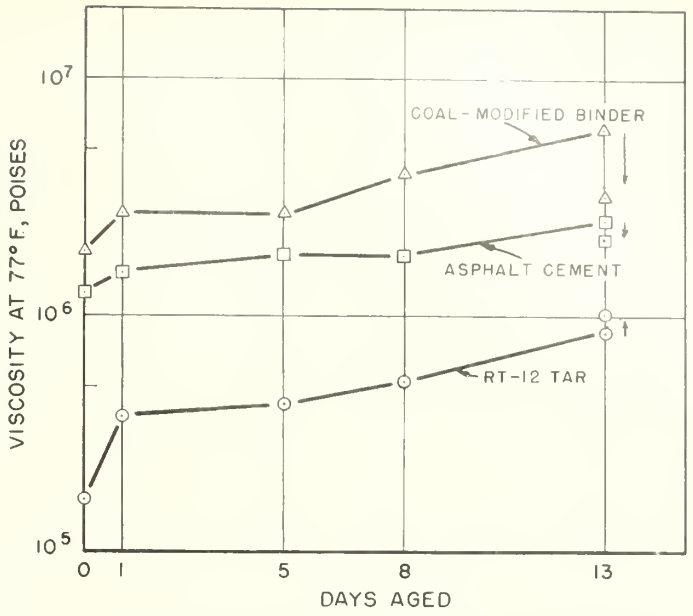

Figure 6.—Effect of aging at 110° F. on binder viscosity.

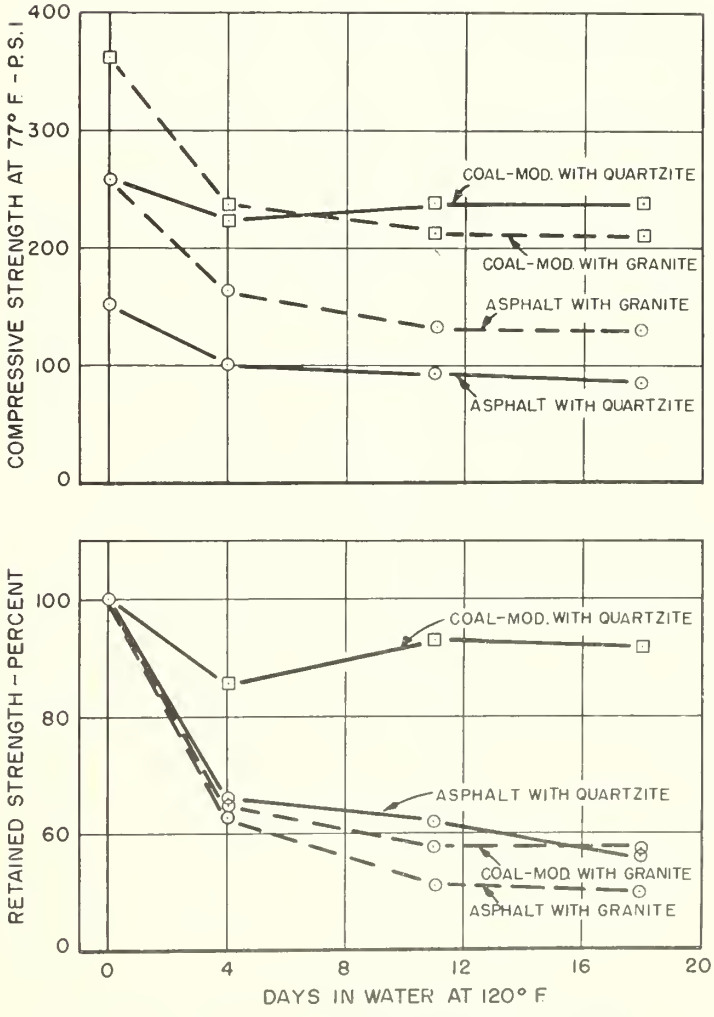

Figure 7.—Effect of water immersion on stability.

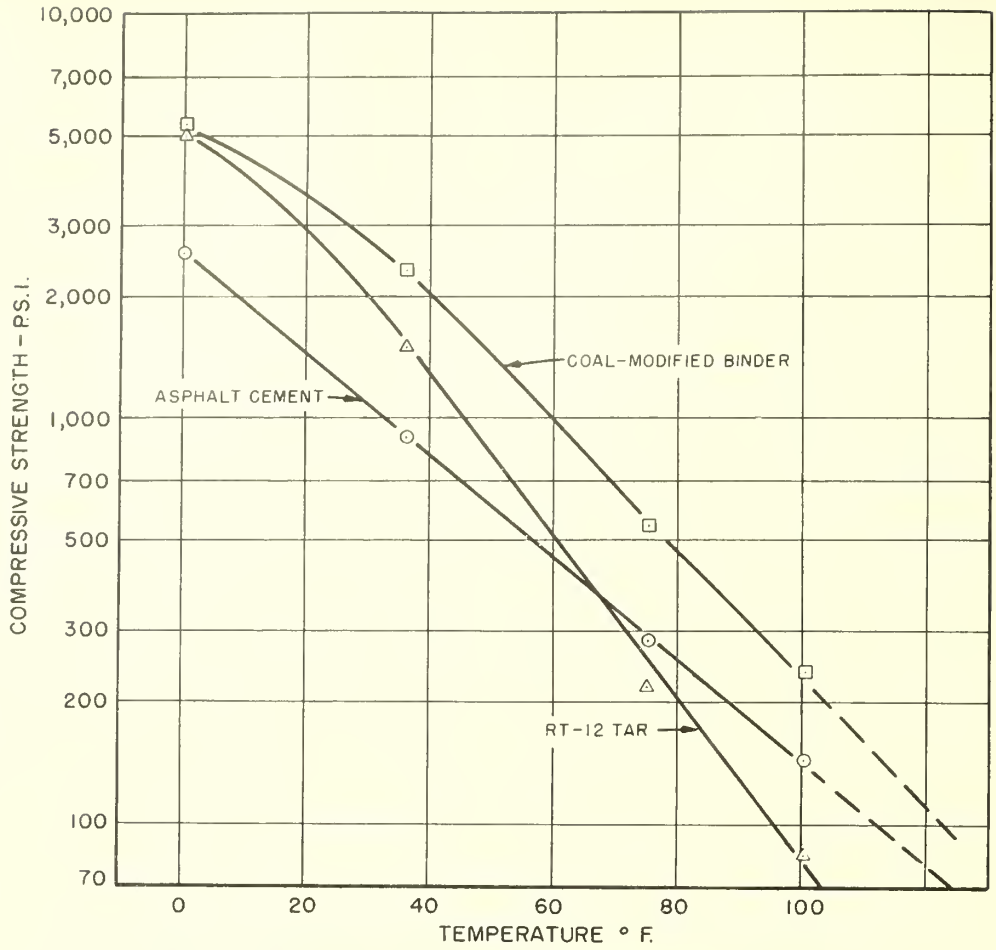

Figure 8.—Variation of compressive strength with temperature.

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Comparison of the Splitting Tensile Strength of Concrete with Flexural and Compressive Strengths

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by WILLIAM E. GRIEB and
GEORGE WERNER, Highway Research Engineers

Much interest has been shown in the use of the splitting tensile test for determining the direct tensile strength properties of concrete because of the questionable results sometimes obtained from other tensile tests. The splitting tensile test was developed more than 10 years ago and has been used successfully in other countries, but its use in the United States has been limited. Although American research laboratories are familiar with the splitting tensile test, little research data has been published. Consequently, information on correlation of this test and the more familiar tests, such as the flexural and compressive strength tests, are required for evaluation of the usefulness of this test.

As a step toward meeting the need for evaluation of the splitting tensile test, more than 6,000 concrete specimens were tested in the laboratory of the Bureau of Public Roads to compare the splitting tensile strength test results with those obtained from flexural and compressive strength tests. The concretes used in the tests were prepared with crushed stone, gravel, and lightweight aggregates. An analysis of the results of these tests is presented in this article. Results showed a straightline relation between the splitting tensile strength and the flexural strength. The relation between the splitting tensile and compressive strengths was curvilinear. The maximum size and the type of aggregate used in the concrete mixture had an effect on the ratio of the splitting tensile strength to the flexural and compressive strengths. These tests also showed that the splitting tensile strengths are not affected as much as the flexural strengths by the moisture condition of the specimens at the time of testing.

Introduction

A RELATIVELY simple test for determining the tensile strength of concrete was devised about 15 years ago; it was developed independently in Japan by Akazawa (*1*)[2] and in Brazil by Carneiro and Barcellos (*2*). This test is known as the splitting tensile or the indirect tensile test. Because it has a number of advantages over the beam test for flexural strength or the direct tensile test on cylinders, this test has been received with favor in the United States for use in determining the tensile strength of concrete. The splitting tensile test usually is made on a 6- by 12-inch cylinder and no capping or grinding of bearings is necessary when proper molds are used, and special grips are not required. The breaks at the failure of the specimen are through the vertical diametral plane and the location of the break does not change as it does in the flexural beam test or the direct tensile test. Furthermore, the specimens are usually smaller and less susceptible to damage than the specimens used for the other two types of tension tests. Also, moisture content of the splitting tensile cylinder has less effect on the tensile strength than moisture content of a concrete beam has on flexural strength. A standard method for making the splitting tensile test has been proposed by the ASTM Committee, C–9, on Concrete and Concrete Aggregates.

Although an appreciable number of laboratories in the United States have used the splitting tensile test, most of the published data about it have been developed in Europe. Wright (*3*) and Thaulow (*4*) concluded from their studies that splitting tensile strength is affected less by the moisture content of the concrete than flexural strength, and that the splitting tensile test provides more uniform results than other types of tensile tests. Test results indicated that splitting tensile strengths are about one-and-a-half times greater than those obtained from direct tensile tests and about two-thirds of those obtained from flexural tests.

Two investigators in the United States recently published separate reports on results of the splitting tensile test. Mitchell (*5*) evaluated the splitting tensile test as a measure of the tensile strength of concrete. He also discussed the different theoretical considerations of failures of brittle materials and concluded that the Mohr theory is a satisfactory means of expressing failure conditions in this test. Hanson (*6*) suggested the use of a combination of the compressive strength and splitting tensile strength tests to determine the resistance of lightweight concrete for structures to shear and diagonal tension. He reported that the splitting tensile strength correlates with the diagonal tension or shear capacity of lightweight concrete in beams loaded to failure. He further indicated that the flexural strength test results can be erratic when moisture distribution in beams is not uniform and that, therefore, flexural strength cannot be correlated directly with the load performance of concrete in structural members. The thought was expressed that the nonuniform distribution of moisture in concrete prepared for tests does not affect the uniformity of either splitting tensile strengths or compressive strengths as much as it affects flexural strengths.

Tests by the Bureau of Public Roads

Tests have been made in the laboratory of the Bureau of Public Roads during the 10-year period, 1951–1961, to determine the relation shown between the splitting tensile, flexural, and compressive strengths of many

[1] Presented at the 65th annual meeting of the American Society for Testing and Materials, New York, N.Y., June 1962.

[2] References indicated by italic numbers in parentheses are listed on page 106.

concretes. During this period, more than 2,000 tests of each type were made. The major variables in these tests were: the type and size of coarse aggregate, the cement content, the moisture content of specimens when they were tested, and the age of the concrete at time of the test.

The specimens were prepared and tested in accordance with the applicable ASTM methods and, except when so noted, were continuously moist cured until test. The splitting tensile and compressive tests were made on 6- by 12-inch cylinders, and the flexural tests were made on 6- by 6- by 21-inch beams that were loaded at the third points. All specimens were cast in metal molds.

Most of the splitting tensile tests were made in connection with other investigations; consequently, materials, mixes, and ages of concrete differed greatly. Twelve different brands of cement and four different siliceous sands that had fineness moduli ranging from 2.60 to 3.00 were used in the tests. The age of specimens at time of test ranged from 7 to 365 days, and the cement content ranged from 4.0 to 8.0 bags per cubic yard of concrete. To develop comparative data on tensile, flexural, and compressive strength test results, one specimen for each type of test was made from a single batch of concrete; these specimens were cured in the same manner and tested at the same age.

Conclusions

The results of tests made in the laboratory of the Bureau of Public Roads warrant the following conclusions.

For a given coarse aggregate and method of curing, a linear relation exists between the splitting tensile strength and the flexural strength of concrete. The relation between the splitting tensile strength and the compressive strength of concrete is curvilinear.

The relation between splitting tensile strength and flexural strength differs according to the type and maximum size of the coarse aggregate used. The relation between splitting tensile strength and compressive strength also differs according to the type and maximum size of the coarse aggregate used.

For a given coarse aggregate and method of curing, the ratio of the splitting tensile strength to the flexural strength is constant, and this relation is not affected by either the cement content of the concrete or the age at test. The ratio of the splitting tensile strength to the compressive strength decreases as the compressive strength increases; therefore, this ratio is affected by both the concrete's cement content and the age at test.

For moist-cured specimens, the splitting tensile strength averaged approximately five-eighths of the flexural strength for gravel concrete, two-thirds of the flexural strength for limestone concrete, and three-fourths of the flexural strength for lightweight aggregate concrete. Similar results are not given for the splitting tensile and compressive strengths because of the nonlinear relation that existed between these strengths.

The splitting tensile strength of the concrete was affected less by drying than the flexural strength. This effect was more pronounced for concrete prepared with lightweight aggregates than for concrete made with natural aggregates. The reduction in splitting tensile strength caused by drying was greater than the reduction in compressive strength of the concrete.

No appreciable difference existed between the unit splitting tensile strength of 6- by 6-inch and 6- by 12-inch cylinders.

Description of Test

A brief description of the method used by Public Roads to make the splitting tensile test follows. To avoid excessive repetition, the splitting tensile test is referred to as the "splitting" test. A 6- by 12-inch cylinder was placed horizontally between the bearing block on the platen and the upper spherically-seated bearing block of a compression testing machine

Table 1.—Comparison of splitting tensile strength with flexural and compressive strengths of concrete containing 1½-inch crushed stone [1]

Splitting strength (S)	Flexural strength (F)	Ratio S to F	Compressive strength (C)	Ratio S to C
P.s.i.	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
180	350	51	1,390	12.9
185	350	53	1,350	13.7
210	355	59	1,470	14.3
255	410	62	1,960	13.0
255	380	67	1,710	14.9
280	420	67	2,330	12.0
285	475	60	2,500	11.4
340	475	72	2,730	12.5
345	445	78	2,720	12.7
345	535	64	2,860	12.1
360	530	68	3,250	11.1
360	555	65	3,160	11.4
370	520	71	2,980	12.4
395	590	67	3,710	10.6
395	510	77	3,250	12.2
410	660	62	3,780	10.8
415	640	65	3,540	11.7
430	630	68	5,070	8.5
430	600	72	3,810	11.3
430	640	67	3,700	11.6
430	645	67	3,520	12.2
435	670	65	3,860	11.3
445	630	71	3,670	12.1
465	805	58	4,570	10.2
500	730	68	5,320	9.4
500	800	63	4,610	10.8
505	730	69	5,620	9.0
505	740	68	5,400	9.4
515	790	65	4,460	11.5
525	------	------	4,990	10.5
530	785	68	6,050	8.8
530	750	71	6,050	8.8
535	850	63	6,940	7.7
540	------	------	5,210	10.4
555	------	------	6,010	9.2
560	750	75	5,790	9.7
560	890	63	6,200	9.0
565	855	66	6,730	8.4
565	875	65	5,580	10.1
565	790	72	6,720	8.4
565	955	59	6,270	9.0
595	880	68	5,940	10.0
595	775	77	6,150	9.7
600	805	75	5,660	10.6
605	------	------	6,090	9.9
620	925	67	7,370	8.4
625	885	71	7,250	8.6
635	875	73	7,210	8.8
Average ratios		67	--------	10.7

[1] Each strength was the average result for five tests. Specimens were stored in moist air until tested. Cement content ranged from 4 to 7½ bags per cubic yard and age at test ranged from 7 to 365 days.

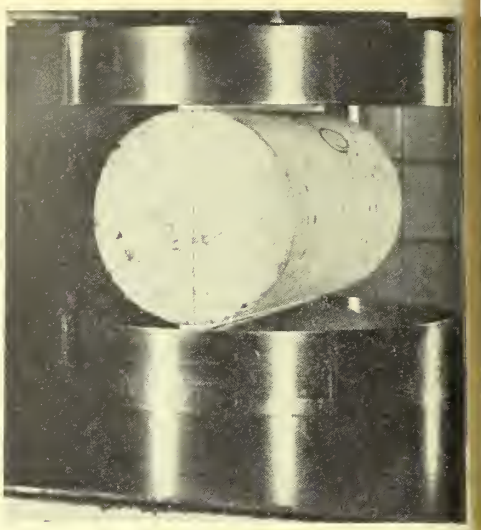


Figure 1.—Cylinder in testing machine for splitting tensile test.

so that the bearing load was applied to opposite elements of the cylinder. Strips of plywood, about one-eighth of an inch thick, three-fourths of an inch wide, and twelve inches long, were placed on the upper and lower bearing elements of the cylinder to ensure uniform bearing pressure. The cylinder was positioned so that the center of its upper bearing element coincided with the center of the upper bearing block of the testing machine. Figure 1 shows a cylinder positioned in the testing machine prior to being loaded. The load was applied at the rate of 150 p.s.i. per minute. In the proposed ASTM method, the load is to be applied at a rate in the range of 100 to 200 p.s.i. per minute or approximately 11,000 to 23,000 pounds per minute for a 6- by 12-inch cylinder. When the cylinder failed, it split through the center and little shattering occurred. A typical break is shown in figure 2.

The following formula [3] was used to calculate the splitting tensile strength of the specimen:

$$T=\frac{2P}{\pi\, ld}$$

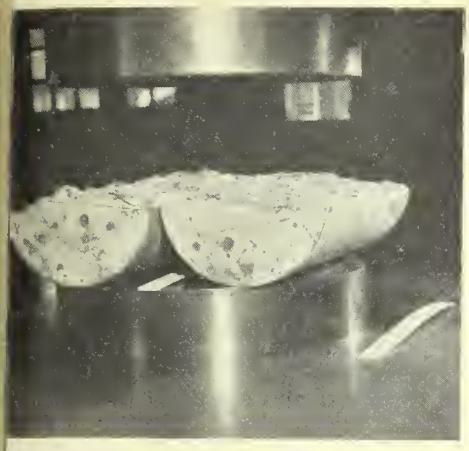
Where,

T = Splitting tensile strength, p.s.i.
P = Maximum applied load at failure pounds.
l = Length of cylinder, inches.
d = Diameter of cylinder, inches.

Effect of Type of Coarse Aggregate

A study was made to determine the effect that the type of coarse aggregate has on the relation of splitting strength to the flexural and compressive strengths of concrete. Specimens were made from concretes prepared with a crushed limestone from a single source, a gravel from a single source, and lightweight fine and coarse aggregates from 10 different sources. When natural sand was used, it was obtained from a single source.

[3] For derivation of formula, see reference to Wright's article (*3*).



igure 2.—Typical break in splitting tensile test.

oncrete prepared with crushed limestone

The splitting, flexural, and compressive rengths of 48 concrete mixtures prepared ith a crushed limestone having a maximum ze of 1½ inches are shown in table 1. The sults have been tabulated in order of ascend- g splitting strengths. The cement content of is concrete ranged from 4 to 7½ bags per ubic yard, and the age of the specimens at me of test was from 7 to 365 days; therefore, wide range in strengths resulted. The plitting-flexural and splitting-compressive rength ratios of identical concretes, expressed s percentages, also are given in table 1. The

able 2.—Comparison of splitting tensile strength with flexural and compressive strengths of concrete containing 1½-inch gravel [1]

Splitting strength (S)	Flexural strength (F)	Ratio S to F	Compressive strength (C)	Ratio S to C
P.s.i.	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
150	250	60	1,180	12.7
250	450	56	1,960	12.8
260	410	63	2,130	12.2
270	400	68	2,330	11.6
280	400	70	1,940	14.4
280	410	68	2,060	13.6
295	505	58	2,680	11.0
300	530	57	2,980	10.1
320	510	63	6,860	11.2
335	525	64	2,600	12.9
340	580	59	3,110	10.9
345	555	62	3,130	11.0
355	540	66	3,440	10.3
355	670	53	3,960	9.0
360	625	58	3,240	11.1
360	670	54	3,670	9.8
365	690	53	3,980	9.2
365	630	58	3,800	9.6
370	740	50	4,100	9.0
375	670	56	3,720	10.1
390	640	61	3,900	10.0
390	570	68	3,360	11.6
405	505	80	3,100	13.1
415	760	55	4,340	9.6
435	635	69	5,300	8.2
435	780	56	4,610	9.4
440	695	63	4,440	9.9
445	780	57	4,600	9.7
465	------	------	4,120	11.3
540	790	68	5,660	9.5
570	790	72	6,660	8.6
Average ratios		62		10.8

[1] Each strength is the average result for two to five tests. Specimens were stored in moist air until tested. Cement content ranged from 4½ to 7½ bags per cubic yard and age at test ranged from 7 to 365 days.

splitting-flexural strength ratios ranged from 51 to 78 percent and the average ratio was 67 percent; the splitting-compressive strength ratios ranged from 7.7 to 14.9 percent and the average ratio was 10.7. As can be observed from the data in table 1, splitting-compressive strength ratios tended to decrease as the compressive strength of the concrete increased. The nonlinear relation between these strengths shows that an average ratio is not applicable throughout the strength range. However, such a ratio serves as a useful index for comparison purposes.

Concrete prepared with gravel

Splitting, flexural, and compressive strengths and the strength ratios of 31 concrete mixtures prepared with a siliceous gravel of 1½-inch maximum size are shown in table 2. The cement contents were from 4½ to 7½ bags per cubic yard and the age of specimens at time of test ranged from 7 to 365 days. The ratios of the splitting strengths to the flexural strengths ranged from 50 to 80 percent and the average ratio was 62 percent. The splitting-compressive strength ratios ranged from 8.2 to 14.4 percent and the average ratio was 10.8.

Concrete prepared with lightweight aggregate

The splitting, flexural, and compressive strengths and the strength ratios of 61 concrete mixtures prepared with lightweight aggregates are shown in table 3. The different fine and coarse lightweight aggregates, including expanded clays, slags, and shales, were used in these tests. Each aggregate was obtained from a different source and the maximum size of the coarse aggregates differed within a range of three-eighths to three-fourths of an inch. The cement contents of the concrete were 6½ and 8 bags per cubic yard, and the ages of the specimens at time of test ranged from 7 to 365 days. The splitting-flexural strength ratios of the lightweight aggregate concrete ranged from 57 to 88 percent and the average ratio was 76 percent; the splitting-compressive strength ratios ranged from 5.3 to 11.2 percent and the average ratio was 8.0 percent.

Relationships for types of coarse aggregate

The relations between the splitting and flexural strengths of concretes prepared with the three types of coarse aggregate—crushed limestone, gravel, and lightweight—are shown in figure 3. The relations were linear, but the slopes differed according to the type of aggregate used. In summary, the average ratio of the splitting strength to the flexural strength was: 67 percent for the concrete made with the crushed limestone, 62 percent for the concrete made with gravel, and 76 percent for the concrete made with the lightweight aggregate.

The relations between the splitting and compressive strengths of the concretes prepared with the three types of coarse aggregate are shown in figure 4. These relations also differed according to the type of aggregate used; but, unlike the splitting-flexural relations, they were nonlinear. The average ratio of the splitting strength to the compressive strength for the concrete made with crushed stone was 10.7 percent, for the concrete made with gravel it was 10.8 percent, and for the concrete made with the lightweight aggregate it was 8.0 percent.

Table 3.—Comparison of splitting tensile strength with flexural and compressive strengths of concrete containing lightweight aggregate [1]

Splitting strength (S)	Flexural strength (F)	Ratio S to F	Compressive strength (C)	Ratio S to C
P.s.i.	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
300	445	67	3,190	9.4
310	430	72	3,430	9.0
315	520	61	3,290	9.6
335	460	73	2,980	11.2
340	435	78	3,040	11.2
340	505	67	4,200	8.1
345	445	78	3,570	9.7
350	510	69	4,130	8.5
360	485	74	4,110	8.8
360	540	67	4,290	8.4
360	500	72	4,290	8.4
365	480	76	3,690	9.9
385	575	67	6,800	5.7
385	520	74	4,060	9.5
385	460	84	3,740	10.3
390	535	73	4,330	9.0
390	500	78	3,900	10.0
405	485	84	4,080	9.9
420	565	74	4,800	8.8
420	565	74	5,290	7.9
420	560	75	6,060	6.9
425	580	73	6,060	7.0
425	570	75	4,480	9.5
425	610	70	5,300	8.0
425	610	70	4,870	8.7
425	750	57	6,300	6.7
440	710	62	8,030	5.5
440	570	77	7,830	5.6
445	555	80	4,440	10.0
445	635	70	6,600	6.7
450	550	82	5,100	8.8
450	575	78	5,920	7.6
460	610	75	5,120	9.0
460	530	87	7,020	6.6
470	635	74	7,460	6.3
470	570	82	5,000	9.4
470	635	74	7,850	6.0
470	680	69	7,800	6.0
480	645	74	6,980	6.9
480	610	79	4,880	9.8
485	605	80	5,080	9.5
490	560	88	5,740	8.5
490	630	78	6,930	7.1
490	635	77	7,490	6.5
490	740	66	6,800	7.2
495	670	74	5,740	8.6
495	735	67	7,590	6.5
495	690	72	6,340	7.8
500	635	79	8,610	5.8
515	640	80	7,760	6.6
520	620	84	6,660	7.8
520	645	81	5,790	9.0
525	680	77	9,870	5.3
530	640	83	8,790	6.0
530	685	77	7,630	6.9
530	650	82	6,760	7.8
540	630	86	6,350	8.5
540	660	82	9,060	6.0
555	710	78	8,790	6.3
565	650	87	7,730	7.3
605	705	86	6,840	8.8
Average ratios		76		8.0

[1] Each strength is the average result of three tests. Specimens were stored in moist air until tested. Cement content was 6½ or 8 bags per cubic yard and age at test ranged from 7 to 365 days.

Effect of Size of Coarse Aggregate

Splitting, flexural, and compressive strength data obtained from tests on concrete made with crushed limestone of 1-inch maximum size were compared with the data given in

Table 4.—Comparison of splitting tensile strength with flexural and compressive strengths of concrete containing 1-inch crushed stone [1]

Splitting strength (S)	Flexural strength (F)	Ratio S to F	Compressive strength (C)	Ratio S to C
P.s.i.	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
450	640	70	4,120	10.9
475	690	69	5,230	9.1
475	640	74	5,330	8.9
490	675	73	4,350	11.3
495	650	76	4,680	10.6
495	695	71	5,150	9.6
500	720	69	-----	-----
505	775	65	5,280	9.6
505	740	68	5,640	9.0
505	760	66	5,380	9.4
510	695	73	5,020	10.2
515	755	68	5,500	9.4
515	690	75	5,650	9.1
520	750	69	5,320	9.8
520	755	69	6,010	8.7
520	695	75	4,900	10.6
525	765	69	5,600	9.4
525	705	74	5,010	10.5
525	710	74	5,560	9.4
530	735	72	5,100	10.4
535	720	74	5,110	10.5
535	730	73	5,530	9.7
540	690	78	5,410	10.0
540	730	74	5,620	9.6
545	740	74	5,770	9.4
545	715	76	5,890	9.3
545	675	81	5,610	9.7
545	740	74	5,350	10.2
550	780	71	6,760	8.1
550	735	75	5,770	9.5
555	740	75	5,320	10.4
555	745	74	5,730	9.7
555	740	75	5,330	10.4
555	800	69	5,810	9.6
560	755	74	5,500	10.2
560	795	70	5,980	9.4
565	790	72	5,940	9.5
565	735	77	5,100	11.1
565	705	80	5,540	10.2
565	790	72	6,110	9.2
565	840	67	5,980	9.4
570	800	71	5,760	9.9
575	830	69	6,160	9.3
575	820	70	5,830	9.9
580	740	78	6,200	9.4
585	765	76	6,010	9.7
595	755	79	5,910	10.1
595	785	76	5,640	10.5
605	800	76	5,940	10.2
615	775	79	6,450	9.5
620	700	89	6,140	10.1
625	810	77	6,050	10.3
Average ratios		73	--------	9.8

[1] Each strength is the average result of three to five tests. Specimens were stored in moist air until tested. Cement content ranged from 5¼ to 6 bags per cubic yard and age at test was 28 days.

table 1 for concrete prepared with the same type of coarse aggregate but having a maximum size of 1½ inches. The results of strength tests made at 28 days on concrete specimens prepared with the crushed stone having a 1-inch maximum size and the calculated strength ratios are shown in table 4. These tests were made on specimens from 52 mixes that had been prepared with 26 different admixtures and with cement contents that ranged from 5¼ to 6 bags per cubic yard. The single age of the specimens and the limited range in their cement content caused smaller differences in strengths than were obtained for specimens prepared with limestone having a maximum size of 1½ inches.

The splitting-flexural strength ratios of the concrete containing the 1-inch crushed stone ranged from 65 to 89 percent and the

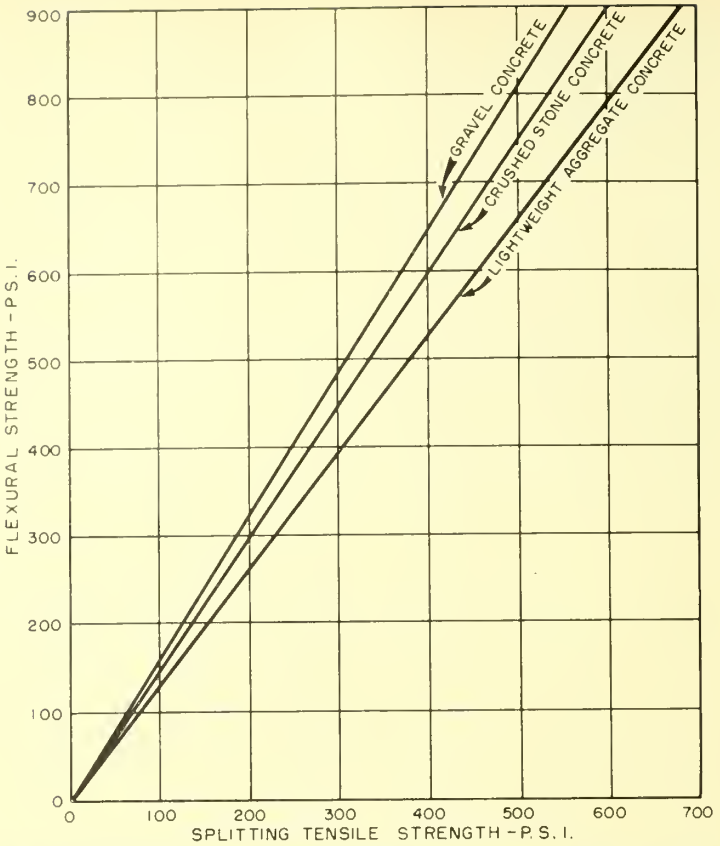


Figure 3.—Relation between flexural and splitting tensile strengths for concrete made with three types of aggregate.

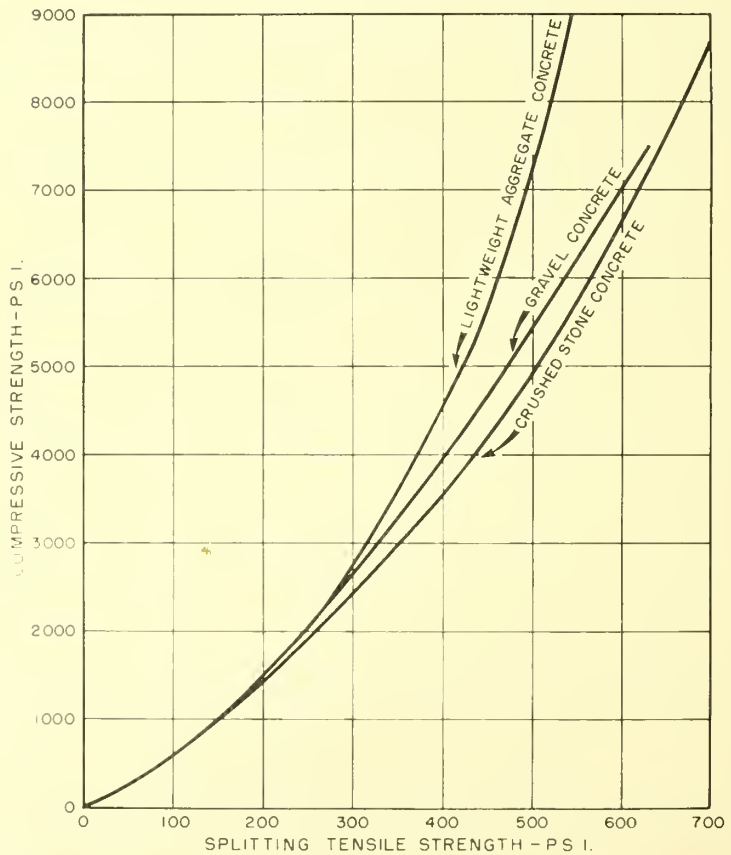


Figure 4.—Relation between compressive and splitting tensile strengths for concrete made with three types of aggregate.

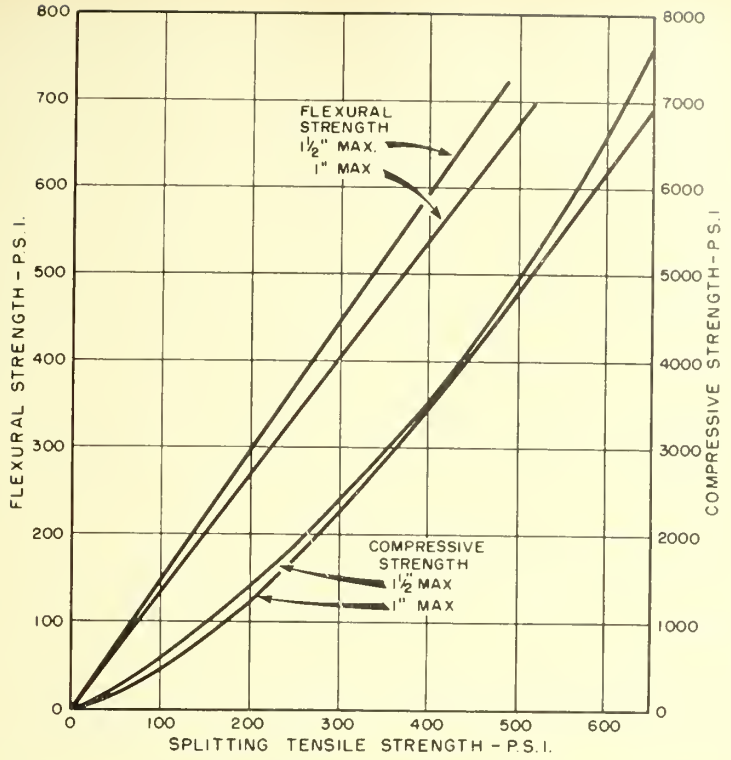


Figure 5.—Effect of size of aggregate on relation of splitting tensile to flexural and compressive strengths.

Table 5.—Effect of drying on splitting tensile, flexural, and compressive strengths of concrete containing lightweight aggregate and tested at 28 days [1]

Curing [2]	Splitting strength (S)	Flexural strength (F)	Ratio S to F	Compressive strength (C)	Ratio S to C	Ratio of strength of dry specimens to strength of moist cured specimens [3]		
						Splitting	Flexural	Compressive
	P.s.i.	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*	*Percent*	*Percent*	*Percent*
Moist	345	445	78	3,570	9.7	------	------	------
Dry	245	210	117	3,350	7.3	71	47	94
Moist	360	485	74	4,110	8.8	------	------	------
Dry	280	220	127	3,770	7.4	78	45	92
Moist	420	565	74	5,290	7.9	------	------	------
Dry	295	210	140	5,120	5.8	70	37	97
Moist	425	750	57	6,300	6.7	------	------	------
Dry	320	210	152	5,680	5.6	75	28	90
Moist	425	585	73	6,060	7.0	------	------	------
Dry	285	180	158	5,740	5.0	67	31	95
Moist	425	610	70	4,870	8.7	------	------	------
Dry	350	265	132	4,730	7.4	82	43	97
Moist	450	550	82	5,100	8.8	------	------	------
Dry	300	220	136	4,550	6.6	67	40	89
Moist	460	610	75	5,120	9.0	------	------	------
Dry	290	245	118	5,550	5.3	63	40	107
Moist	490	560	88	5,740	8.5	------	------	------
Dry	330	205	161	5,480	6.0	67	37	95
Moist	495	670	74	5,740	8.6	------	------	------
Dry	330	190	174	5,530	6.0	67	28	96
Moist	495	690	72	6,340	7.8	------	------	------
Dry	345	265	130	6,510	5.3	70	38	103
Moist	520	645	81	5,790	9.0	------	------	------
Dry	375	295	127	5,820	6.4	72	46	101
Moist	530	685	77	7,620	7.0	------	------	------
Dry	325	260	125	7,080	4.6	61	38	93
Moist	540	630	86	6,350	8.5	------	------	------
Dry	310	220	141	5,740	5.4	57	35	90
Average:								
Moist	455	605	76	5,570	8.3	------	------	------
Dry	315	230	138	5,330	6.0	69	38	96

[1] Each strength is the average result of three tests. Cement content was 6½ or 8 bags per cubic yard.
[2] Moist specimens were stored in moist air at 73° F. continuously for 28 days. Dry specimens were stored in moist air for 7 days, followed by 21 days in laboratory air at 73° F. and 50 percent relative humidity.
[3] Ratio of the strength of dry specimens to the strength of the corresponding moist cured specimens.

average ratio was 73 percent. The splitting-compressive strength ratios ranged from 8.1 to 11.3 percent and the average ratio was 9.8 percent. The corresponding average strength ratios of the concrete containing the 1½-inch crushed limestone were 67 and 10.7 percent, respectively. The splitting-flexural and splitting-compressive strength relations for the concrete containing 1-inch and 1½-inch crushed limestone aggregate are shown in figure 5. It is evident that the maximum size of the coarse aggregate only had a slight effect on these strength relations.

Effect of Drying on Lightweight Aggregate Concrete

Tests were made at 28 and 365 days to determine the effect of drying on the splitting, flexural, and compressive strengths of concrete prepared with lightweight aggregates. One-half of the specimens tested at 28 days was given 7 days of moist curing at 73° F., which was followed by 21 days of storage in laboratory air at 73° F. and 50 percent relative humidity; the other half of the specimens was moist cured continuously. One-half of the specimens tested at 365 days was given 7 days of moist curing, which was followed by 358 days of storage in laboratory air; the other half was moist cured continuously. Differences in the aggregates used and cement contents—6½ and 8 bags per cubic yard of concrete—caused a wide range in strengths. The strength results of the tests at 28 days and the ratios of splitting-flexural and splitting-compressive strengths are shown in table 5. The last three columns of the table contain data showing the splitting, flexural, and compressive strength ratios of the dry specimens (7 days moist cured and then dried in laboratory air) to the wet specimens (continuously moist cured). Similar data obtained from the tests at 365 days are shown in table 6.

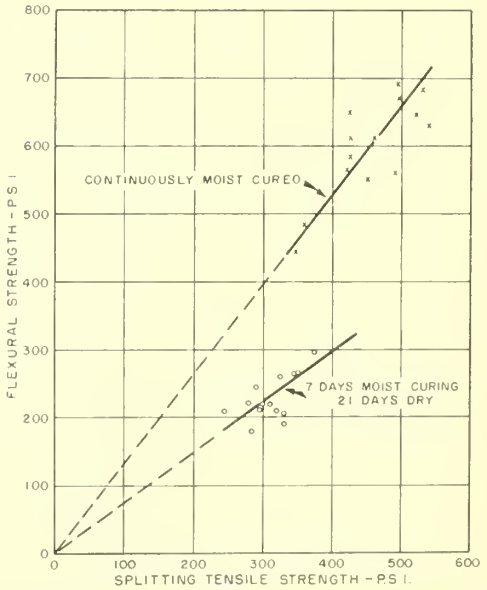


Figure 6.—Effect of drying on relation of flexural and splitting tensile strengths of concrete containing lightweight aggregate, at 28 days.

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Table 6.—Effect of drying on splitting tensile, flexural, and compressive strengths of concrete containing lightweight aggregate and tested at 365 days [1]

Curing [2]	Splitting strength (S)	Flexural strength (F)	Ratio S to F	Compressive strength (C)	Ratio S to C	Ratio of strength of dry specimens to strength of moist cured specimens [3]		
						Splitting	Flexural	Compressive
	P.s.i.	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*	*Percent*	*Percent*	*Percent*
Moist	340	505	67	4,200	8.1	------	------	------
Dry	335	330	102	3,580	9.4	99	65	85
Moist	360	540	67	4,290	8.4	------	------	------
Dry	290	270	107	3,950	7.3	81	50	92
Moist	440	570	77	7,830	5.6	------	------	------
Dry	430	535	80	5,390	8.0	98	94	69
Moist	440	710	62	8,030	5.5	------	------	------
Dry	380	330	115	7,140	5.3	86	46	89
Moist	470	635	74	7,850	6.0	------	------	------
Dry	395	295	134	6,740	5.9	84	46	86
Moist	470	635	74	7,460	6.3	------	------	------
Dry	495	205	241	5,330	9.3	105	32	71
Moist	500	635	79	8,610	5.8	------	------	------
Dry	490	470	104	6,360	7.7	98	74	74
Moist	515	640	80	7,760	6.6	------	------	------
Dry	480	420	114	6,090	7.9	93	66	78
Moist	525	680	77	9,870	5.3	------	------	------
Dry	495	400	124	9,060	5.5	94	59	92
Moist	530	640	83	8,790	6.0	------	------	------
Dry	420	380	111	7,530	5.6	79	59	86
Moist	540	660	82	9,060	6.0	------	------	------
Dry	415	205	202	7,380	5.6	77	31	81
Averages:								
Moist	465	625	75	7,610	6.3	------	------	------
Dry	420	350	130	6,230	7.0	90	57	82

[1] Each strength is the average result of three tests. Cement content was 6½ or 8 bags per cubic yard.
[2] Moist specimens were continuously stored in moist air at 73° F. Dry specimens were stored in moist air for 7 days, followed by 358 days in laboratory air at 73° F. and 50 percent relative humidity.
[3] Ratio of the strength of dry specimens to the strength of the corresponding moist cured specimens.

From the tests at 28 days, the average ratio of splitting-flexural strengths was 76 percent for the wet specimens and 138 percent for the dry specimens. Corresponding ratios for the tests made at 365 days were 75 and 130 percent. Likewise, the average ratio of splitting-compressive strengths for the tests at 28 days was 8.3 percent for the wet specimens and 6.0 percent for the dry specimens. Similar ratios for the tests at 365 days were 6.3 and 7.0 percent. The effect of the moisture content of concrete containing lightweight aggregate on the splitting-flexural and splitting-compressive relations is shown in figures 6–9. The comparative ratios and relations determined in this study emphasize the importance of the effect of moisture content of lightweight aggregate concrete on the splitting-flexural strength relations. The influence of the moisture content on the splitting-compressive strength relations was very pronounced in the results of the tests at 28 days, but no significant influence was indicated in the results of the tests at 365 days.

As stated previously, the last three columns of tables 5 and 6 show the ratios of the strengths of dry specimens to the strengths of the corresponding wet specimens for each of

Table 7.—Effect of drying on splitting tensile, flexural, and compressive strengths of concrete containing crushed stone

Aggregate and curing [1]	Splitting strength (S)	Flexural strength (F)	Ratio S to F	Compressive strength (C)	Ratio S to C	Ratio of strength of dry specimens to strength of moist cured specimens [2]		
						Splitting	Flexural	Compressive
Crushed limestone, maximum size three-fourths inch: [3]	*P.s.i.*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*	*Percent*	*Percent*	*Percent*
28 days moist	575	800	72	5,700	10.1	------	------	------
7 days moist and 21 days dry	485	480	101	5,720	8.5	84	60	100
Crushed limestone, maximum size 1½ inches. [4]								
28 days moist	565	795	71	5,410	10.4	------	------	------
1 day moist and 27 days dry	360	410	88	3,530	10.2	64	52	65
7 days moist and 21 days dry	495	480	103	5,430	9.1	88	60	100
7 days moist, 20 days dry, and 1 day wet	590	515	115	5,120	11.5	104	65	95

[1] Moist cured specimens were stored in moist air at 73° F. Dry specimens were stored in laboratory air at 73° F. and 50 percent relative humidity.
[2] Ratio of the strength of partially moist cured specimens to the strength of the corresponding moist cured specimens.
[3] Each strength is the average result of six tests. Cement content was 6½ or 8 bags per cubic yard.
[4] Each strength is the average result of five tests. Cement content was 6 bags per cubic yard.

Table 8.—Effect of cement content [1] splitting tensile, flexural, and compressive strengths of concrete prepared with different aggregates [1]

Cement content	Splitting strength (S)	Flexural strength (F)	Ratio S to F	Compressive strength (C)	Ratio S to C
Limestone (three-fourths inch maximum size) [2]					
Bags/cu. yd.	*P.s.i.*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
6.5	550	760	72	5,820	9.5
8.0	580	805	72	6,360	9.1
Limestone (1½ inches maximum size) [3]					
4.1	400	595	67	3,480	11.5
6.0	530	760	70	5,550	9.5
7.5	580	870	67	6,630	8.7
Gravel (1½ inches maximum size) [4]					
4.5	280	450	62	2,960	9.5
7.0	400	610	66	4,960	8.1
Lightweight aggregate [5]					
6.5	430	580	74	5,590	7.7
8.0	465	605	77	6,220	7.5

[1] Specimens were stored in moist air at 73° F. until test
[2] Each strength is the average result of 12 tests. Age test ranged from 7 to 365 days.
[3] Each strength is the average result of 100 tests. Age test was 28 days.
[4] Each strength is the average result of 20 tests. Age test was 28 days.
[5] Each strength is the average result of 75 tests. Age test ranged from 7 to 365 days.

the three types of strength tests. For t tests at 28 days, the average ratios were 6 38, and 96 percent for the splitting, flexura and compressive strength tests, respectivel Similar ratios for the tests at 365 days we 90, 57, and 82 percent. Based on individu test ratios at 28 and 365 days, the reduction i splitting strength from 22 of the 25 tests w less than 33 percent; but the reduction i flexural strength from 19 of the 25 tests w 50 percent or more.

In tests at both 28 and 365 days, dry storag of concrete containing lightweight aggregat had an appreciably greater deleterious effe on flexural strength than on splitting strengt This might have been caused by the fin cracks that developed on the surface of th concrete as the flexural test specimens drie Because the exteriors of the test cylinders we under compression, the splitting strength w not affected as much by the surface conditio of the test specimens. Conversely, in tes at 28 days, the compressive strength was a fected less by dry storage than the splittin strength, the average reduction being 31 per cent for splitting strength and 4 percent fo compressive strength. But in tests at 365 days drying caused little difference in the reductio of splitting and compressive strengths o concretes prepared with lightweigh aggregates.

Effect of Drying on Crushed Limestone Concrete

The results of two series of tests made t study the effect of drying on the splitting

Table 9.—Effect of age at test on splitting tensile, flexural, and compressive strengths of concrete [1]

Age at test	Splitting strength (S)	Flexural strength (F)	Ratio S to F	Compressive strength (C)	Ratio S to C
GRAVEL (1½ inches maximum size) [2]					
Days	*P.s.i.*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
7	270	450	60	2,360	11.4
14	350	590	59	3,410	10.3
28	375	610	61	3,370	11.1
LIGHTWEIGHT AGGREGATE [3]					
7	385	510	75	4,030	9.6
28	455	610	75	5,610	8.1
90	470	630	75	6,760	7.0
365	460	610	75	7,450	6.2
LIMESTONE (three-fourths inch maximum size) [4]					
7	510	740	69	4,740	10.8
28	575	800	72	5,700	10.1
90	540	805	67	6,700	8.1

[1] Specimens were stored in moist air at 73° F. until tested.
[2] Each strength is the average result of 15 tests. Cement content ranged from 4½ to 7½ bags per cubic yard.
[3] Each strength is the average result of 40 tests. Cement content was 6½ or 8 bags per cubic yard.
[4] Each strength is the average result of 6 tests. Cement content was 6½ or 8 bags per cubic yard.

Table 10.—Effect of length of cylinder on splitting tensile strength of concrete [1]

Splitting tensile strength [2]	
6- by 6-in. cylinders	6- by 12-in. cylinders
P.s.i.	*P.s.i.*
270	275
300	300
385	430
430	360
355	390

[1] Specimens were stored in moist air at 73° F. until tested. Each strength is the average result of two tests.
[2] Average splitting tensile strength for both sizes of cylinders was 350 p.s.i.

Table 11.—Effect of bearing surface on splitting tensile strength of concrete [1]

Splitting tensile strength	
Plywood bearings [2]	Lumnite cement bearings [3]
P.s.i.	*P.s.i.*
535	515
540	535
575	640
540	570
440	430
445	445
440	440
430	440

[1] Specimens were stored in moist air at 73° F. until tested. Each strength is the average of three tests.
[2] Average splitting strength was 495 p.s.i.
[3] Average splitting strength was 500 p.s.i.

Table 12.—Comparison of uniformity of splitting strength with flexural and compressive strengths of concrete prepared with crushed limestone of 1-inch maximum size [1]

Batch number	Splitting strength		Flexural strength		Compressive strength	
		Variation from average		Variation from average		Variation from average
	P.s.i.	*Percent*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
1	555	1.3	760	−3.1	5,690	2.9
	555	1.3	720	−8.2	5,540	0.2
2	550	0.4	775	−1.1	5,570	0.7
	515	−6.0	845	7.8	5,620	1.6
3	545	−0.5	795	1.4	5,500	−0.5
	515	−6.0	705	−10.1	5,440	−1.6
4	530	−3.3	850	8.4	5,570	0.7
	560	2.2	805	2.7	5,640	2.0
5	545	−0.5	685	−12.6	5,420	−2.0
	540	−1.5	750	−4.3	5,390	−2.5
6	565	3.1	840	7.1	5,610	1.4
	515	−6.0	770	−1.8	5,560	0.5
7	560	2.2	795	1.4	5,330	−3.6
	545	−0.5	685	−12.6	5,480	−0.9
8	545	−0.5	780	−0.5	5,510	−0.4
	535	−2.4	760	−3.1	5,420	−2.0
9	535	−2.4	765	−2.4	5,330	−3.6
	530	−3.3	860	9.7	5,550	0.4
10	605	10.4	835	6.5	5,230	−5.4
	510	−6.9	820	4.6	5,210	−5.8
11	560	2.2	740	−5.6	5,310	−4.0
	560	2.2	735	−6.2	5,300	−4.2
12	550	0.4	805	2.7	5,630	1.8
	550	0.4	805	2.7	5,660	2.4
13	530	−3.3	810	3.3	5,650	2.2
	540	−1.5	820	4.6	5,620	1.6
14	495	−9.7	900	14.8	6,010	8.7
	580	5.8	785	0.1	5,720	3.1
15	630	15.0	760	−3.1	5,700	3.1
	575	4.9	770	−1.8	5,710	3.3
AVERAGE	548	3.5	784	5.1	5,530	2.4
Coefficient of variation, percent		5.0		7.2		3.1

[1] Specimens were stored in moist air at 73° F. until tested. Age at test was 28 days and cement content was 6 bags per cubic yard.

flexural, and compressive strengths of concrete prepared with a crushed limestone coarse aggregate are shown in table 7. The first series of tests produced results similar to those obtained in tests at 28 days on the lightweight aggregate concrete. The second series of tests was made at 28 days on four different groups of specimens; for each group, the specimens were cured by a different combination of alternating moist and dry storage.

First series

In the first series of tests, the maximum size of the crushed stone was three-fourths of an inch, which was the same maximum size as some of the lightweight aggregates used. The results showed that drying caused average strength losses of 16, 40, and 0 percent for the splitting, flexural, and compressive tests, respectively. The corresponding strength losses for the lightweight aggregate concrete, shown in table 5, were 31, 62, and 4 percent. In general, the comparison of the data from tests at 28 days indicates that drying caused greater strength losses in concrete prepared with lightweight aggregate than in the concrete prepared with the limestone coarse aggregate. The difference in strength loss was greater for the flexural than the splitting test and was insignificant for the compressive test.

Second series

In the second series of tests, the maximum size of the crushed stone used was 1½ inches. For each type of strength test, 20 specimens were made: (1) Five control specimens were moist cured continuously; (2) five specimens were moist cured for 1 day, then were stored in laboratory air for 27 days; (3) five specimens were moist cured for 7 days, then were stored in laboratory air for 21 days; and (4) five specimens were moist cured for 7 days, were stored in laboratory air for 20 days, and then were immersed in water for 1 day. The data, given in table 7, show that the flexural strength was affected more by drying than the splitting or compressive strengths. The losses in splitting and compressive strengths caused by drying were approximately the same. Comparisons between similarly cured concretes prepared with crushed stone aggregate having maximum sizes of ¾ and 1½ inches are also shown in table 7. Of particular note is the fact that in each of the three strength tests, the strength losses caused by the same drying conditions were nearly identical for concretes prepared with the two sizes of coarse aggregate.

COLLATERAL STUDIES

In conjunction with the research program that has been described, additional data of interest and value are discussed in the paragraphs that follow.

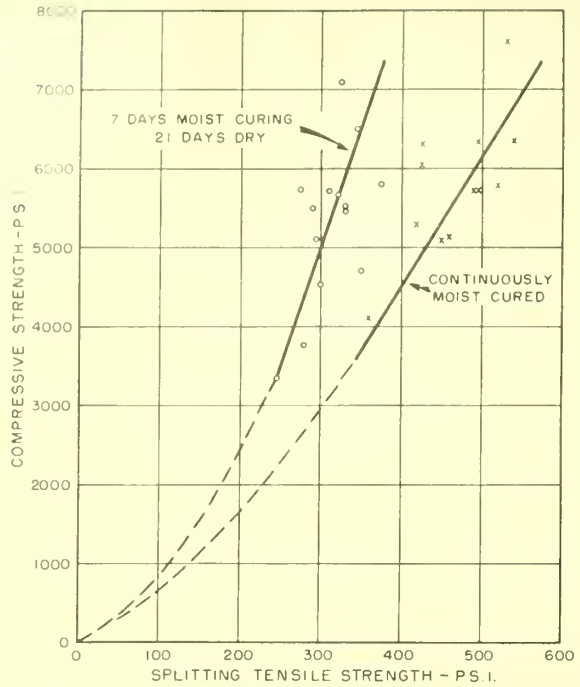

Figure 7.—Effect of drying on relation of compressive and splitting tensile strengths of concrete containing lightweight aggregate, at 28 days.

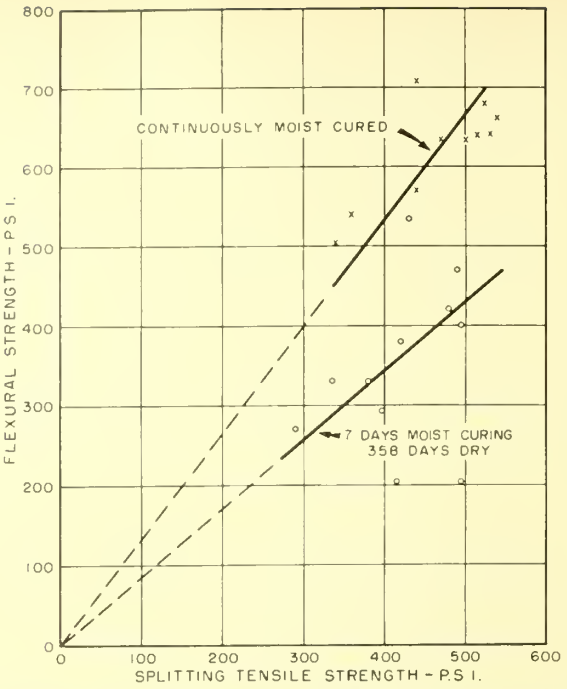

Figure 8.—Effect of drying on relation of flexural and splitting tensile strengths of concrete containing lightweight aggregate, at 365 days.

Cement Content

At a number of places in the article mention has been made that the cement content of the concrete was different in the test specimens. The effect of cement content on splitting-flexural and splitting-compressive strength relations is shown by the data in table 8. They indicate that the range in cement content used in this investigation had little influence on the splitting-flexural ratios of concrete prepared with the same type and maximum size of coarse aggregate. However, the splitting-compressive ratios decreased as the cement content of the concrete was increased for each group of comparative specimens.

Age of Concrete at Test

The effect of age of the concrete at test on the splitting-flexural and splitting-compressive strength relations is shown by data in table 9. They were obtained from several groups of specimens for which the cement content of the concrete was different in each group. To minimize the influence of the cement content, the same number of specimens for each cement content was tested at each of the indicated ages. The data in table 9 show that no appreciable difference in splitting-flexural strength ratios occurred for concrete prepared with the same type and maximum size of coarse aggregate; but, for each group of specimens, the splitting compressive strength ratios decreased as the age at test increased.

Length of Test Cylinder

In the main research program, tests were made only on 6- by 12-inch cylinders. To determine whether the length of the cylinder

Table 13.—Comparison of uniformity of splitting strength with flexural and compressi strengths of concrete made with lightweight aggregate and moist cured [1]

Batch number	Splitting strength		Flexural strength		Compressive strength	
		Variation from average		Variation from average		Variation from average
	P.s.i.	*Percent*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
1	355	− 1.9	480	1.3	2,940	−7.8
	340	− 6.1	465	− 1.9	3,240	1.6
2	290	−19.9	490	3.4	3,460	8.5
	350	− 3.3	555	17.1	3,320	4.1
3	400	10.5	510	7.6	3,270	2.6
	415	14.6	480	1.3	3,220	1.0
4	340	− 6.1	500	5.5	2,980	−6.5
	390	7.7	480	1.3	3,060	−4.0
5	365	0.8	445	− 6.1	3,120	−2.1
	410	13.3	445	− 6.1	3,110	−2.4
6	390	7.7	455	− 4.0	3,190	0.1
	360	− 0.6	480	1.3	3,340	4.8
7	360	− 0.6	490	3.4	3,260	2.3
	415	14.6	460	− 3.0	3,420	7.3
8	375	3.6	465	− 1.9	3,270	2.6
	335	− 7.5	500	5.5	3,220	1.0
9	295	−18.5	475	0.2	3,290	3.2
	335	− 7.5	480	1.3	3,260	2.3
10	380	5.0	410	−13.5	3,150	−1.2
	390	7.7	455	− 4.0	3,110	−2.4
11	375	3.6	470	− 0.8	3,200	0.4
	350	− 3.3	475	0.2	3,120	−2.1
12	335	− 7.5	480	1.3	3,260	2.3
	365	0.8	430	− 9.3	3,190	0.1
13	370	2.2	455	− 4.0	3,160	−0.9
	350	− 3.3	510	7.6	3,030	−5.0
14	380	5.0	485	2.3	3,060	−4.0
	310	−14.4	475	0.2	3,130	−1.8
15	375	3.6	465	− 1.9	3,120	−2.1
	350	− 3.3	445	− 6.1	3,130	−1.8
AVERAGE	362	6.8	474	4.1	3,188	−2.9
Coefficient of variation, percent	--------	8.8	--------	5.7	--------	4.3

[1] Specimens were stored in moist air at 73° F. until tests. Age at test was 28 days and cement content was 6 bags cubic yard.

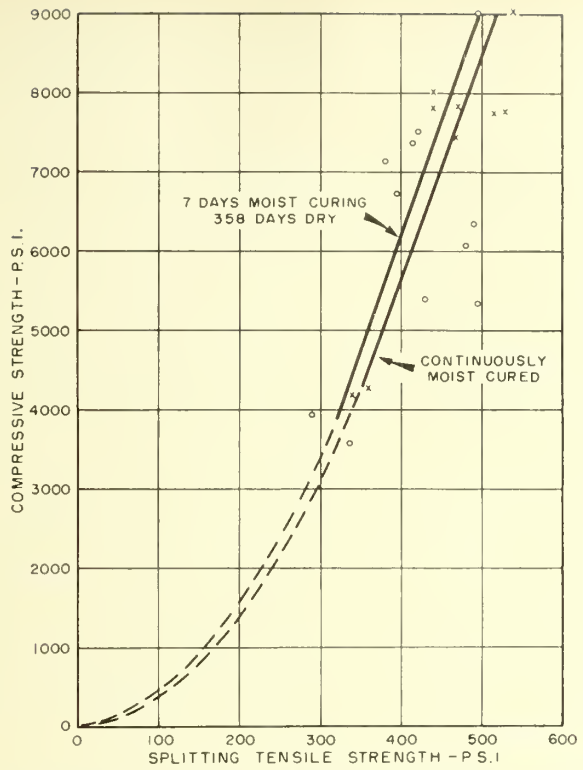

Figure 9.—Effect of drying on relation of compressive and splitting tensile strengths of concrete containing lightweight aggregate, at 365 days.

Table 14.—Comparison of uniformity of splitting strength with flexural and compressive strengths of concrete made with lightweight aggregate and given intermittent curing [1]

Batch number	Splitting strength		Flexural strength		Compressive strength	
		Variation from average		Variation from average		Variation from average
	P.s.i.	*Percent*	*P.s.i.*	*Percent*	*P.s.i.*	*Percent*
1	295	7.7	240	−2.0	2,720	−0.9
	270	−1.5	260	6.1	2,910	6.0
2	260	−5.1	270	10.2	2,770	0.9
	295	7.7	230	−6.1	2,810	2.4
3	275	0.4	220	−10.2	2,860	4.2
	310	13.1	225	−8.2	2,710	−1.3
4	260	−5.1	250	2.0	2,700	−1.6
	280	2.2	230	−6.1	2,680	−2.4
5	250	−8.8	220	−10.2	2,670	−2.7
	265	−3.3	295	20.4	2,720	−0.9
6	290	5.8	235	−4.1	2,980	8.6
	280	2.2	225	−8.2	2,820	2.7
7	270	−1.5	265	8.2	2,830	3.1
	245	−10.6	225	−8.2	2,840	3.5
8	260	−5.1	225	−8.2	2,420	−11.8
	230	−16.1	235	−4.1	2,510	−8.6
9	275	0.4	245	0.0	2,700	−1.6
	290	5.8	255	4.1	2,770	0.9
10	305	11.3	265	8.2	2,700	−1.6
	300	9.5	260	6.1	2,580	−6.0
11	310	13.1	270	10.2	2,770	0.9
	285	4.0	230	−6.1	2,890	5.3
12	250	−8.8	255	4.1	2,730	−0.5
	255	−6.9	245	0.0	2,740	−0.2
13	300	9.5	245	0.0	2,830	3.1
	270	−1.5	260	6.1	2,570	−6.4
14	245	−10.6	245	0.0	2,820	2.7
	270	−1.5	215	−12.2	2,820	2.7
15	260	−5.1	250	2.0	2,720	−0.9
	270	−1.5	260	6.1	2,770	0.9
AVERAGE	274	6.2	245	6.3	2,745	3.2
Coefficient of variation, percent	-------	7.6	-------	7.7	-------	4.3

[1] Specimens were stored in moist air at 73° F. for 7 days, followed by 21 days of storage in laboratory air at 73° F. and 50 percent relative humidity. Cement content was 6 bags per cubic yard.

affects the results of splitting tests, 6- by 6-inch and 6- by 12-inch cylinders were made from the same batch of concrete and tested at the same age. No appreciable difference was noted between the strengths obtained in tests of the two different lengths of cylinders. The results of these tests are given in table 10.

Type of Bearing Surface

A limited series of tests was made to determine the effect of the type of bearing material on splitting-tensile strength. Tests were made on similar specimens of concrete; in these tests plywood bearing strips and neat Lumnite cement bearings were used. A metal jig was used so that strips of the neat Lumnite cement, one-half inch wide and one-eighth inch thick, were cast on diametrically opposite elements of the cylinder. The strips were cast against plane plate glass and all specimens were kept moist until tested. As shown in table 11, the two types of bearing surfaces caused no appreciable differences in the strengths obtained in these tests.

Uniformity Tests

Tests were made to determine the uniformity of the splitting strength as compared with the uniformity of the compressive and flexural strengths of similar concrete. For these tests, 15 batches of concrete were made on each of three days, and two specimens for each type of test were prepared from each batch. All batches of concrete were prepared to be as nearly alike as possible. The specimens were tested at an age of 28 days.

On the first mixing day, specimens were made with crushed limestone having a maximum size of 1 inch and were continuously moist cured until tested. The splitting, flexural, and compressive strengths and the variations from the average strengths are given in table 12. The average variation and the coefficient of variation for each type of test are also given in this table. The coefficient of variation for the splitting strength tests was 5.0 percent, for the flexural strength tests it was 7.2 percent, and for the compressive strength tests it was only 3.1 percent.

On the second mixing day, specimens were made with lightweight aggregate having a maximum size of three-fourths of an inch and were continuously moist cured until tested. The results of these tests are shown in table 13. The coefficient of variation for the splitting strength tests was 8.8 percent, for the flexural strength tests it was 5.7 percent, and for the compressive strength tests it was 4.3 percent.

On the third mixing day, specimens were made with lightweight aggregate and were similar to those made on the second day, but these specimens were given 7 days moist curing followed by 21 days of storage in laboratory air. The results of the strength tests on these specimens are given in table 14. The coefficient of variation for the splitting strength tests was 7.6 percent, for the flexural strength tests it was 7.7 percent, and for the compressive strength tests it was 4.3 percent.

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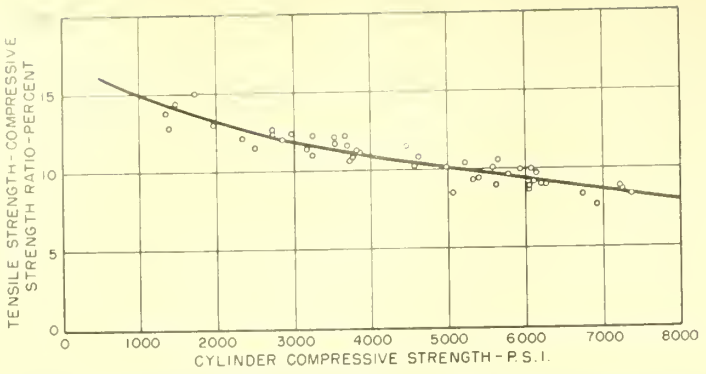

Figure 10.—Relation between ratio of splitting tensile to compressive strength and compressive strength for concrete made with crushed stone of 1½ inches maximum size.

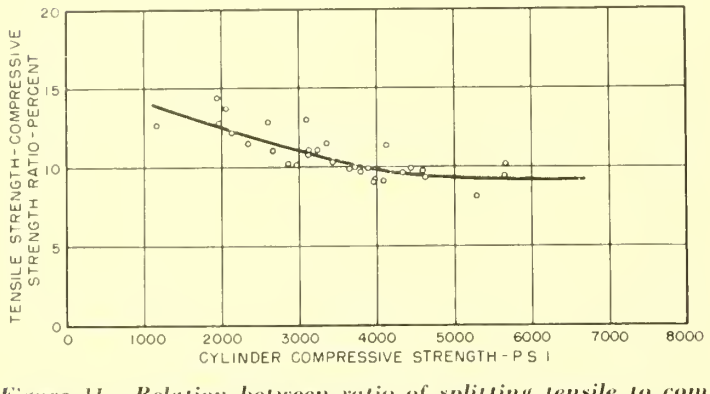

Figure 11.—Relation between ratio of splitting tensile to compressive strength and compressive strength for concrete made with gravel of 1½ inches maximum size.

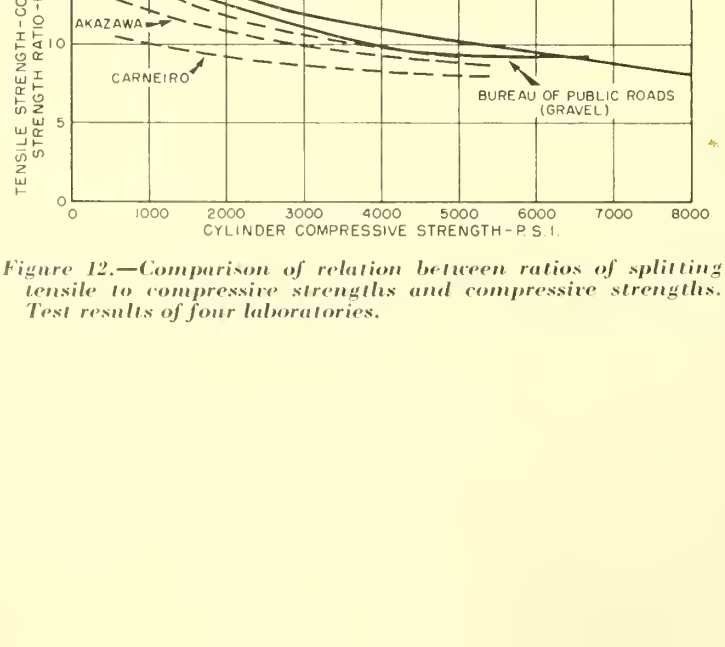

Figure 12.—Comparison of relation between ratios of splitting tensile to compressive strengths and compressive strengths. Test results of four laboratories.

Comparison of Results

The previously mentioned report by Thaulow (*4*) contains a graph, included there as figure 3, that shows a comparison of splitting tensile tests performed in Japan by Akazawa (*1*), in Brazil by Carneiro and Barcellos (*2*), and in Denmark by Efsen and Glarbo (*7*). The data obtained by these investigators were plotted as the relation between the splitting-compressive strength ratio in percentage, and the compressive strength in p.s.i. The data given in tables 1 and 2 of this article have been plotted in a similar manner in figures 10 and 11, respectively; and the relations established are compared in figure 12 with those shown in the Thaulow report. It is apparent that the relations developed by the Bureau of Public Roads are similar to those developed by other investigators. The Bureau's data show that the relation between the splitting-compressive strength ratio and the compressive strength is related to the type of coarse aggregate used in the concrete. It is not known what materials were used by the other investigators.

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The Effect of Expressway Design on Driver Tension Responses

BY THE TRAFFIC OPERATIONS RESEARCH DIVISION BUREAU OF PUBLIC ROADS

Reported [1] by **RICHARD M. MICHAELS**, Research Psychologist

The relationship of highway design to driving stress has been the subject of considerable discussion. The study reported in this article was aimed at measuring driver tension by use of the galvanic skin reflex. Four expressways of differing design were driven. It was found that a freeway with complete control of access and good geometric design generates significantly less driver tension than less rigorous designs. Also, tension is dependent on traffic volume, rising sharply as volume approaches practical capacity. The results do raise the question of whether tension rises because capacity is reached or whether the capacity limitation occurs because at higher volumes tension rises sharply, hence causing the driver to make compensatory responses.

Comparisons of freeways with urban arterials and primaries indicated the latter generated up to four and one-half times as much tension as the freeway. The real benefit comes from the almost total elimination of marginal conflicts for the freeway driver. The results of this study indicate that there may be two factors involved in the concept of comfort and convenience Comfort may reflect the unpredictable interferences in driving: convenience may reflect the predictable interferences such as traffic control devices. If comfort and convenience can be separated, the GSR may be a direct measure of route comfort.

Introduction

A PREVIOUS study conducted by the Bureau of Public Roads (*1*) [2] indicated that driver tension responses, as measured by galvanic skin reflex (GSR), could be used to differentiate between different types of city streets. In the study reported here, the same technique was used in an attempt to determine whether driver tension responses could be used similarly to differentiate between types of design of expressways, and also to determine whether such responses could be used to indicate differences in other types of highways.

With the basic aim of differentiating between expressway designs by using driver tension as the distinguishing measure, two types of tension-inducing events were of prime interest: (1) events of traffic interferences similar to those encountered on urban streets, and (2) events associated with the interferences caused by geometric design features of the highways. Considerable evidence supports the superiority of expressway design over the older highway designs or highways with less control of access. However, it is still a rather moot point as to whether any differences exist among the various philosophies of design that are being proposed for controlled-access highways.

Expressways studied

In the Washington, D.C., metropolitan area, it was possible to find expressway designs of considerably different types, which were distinguishable on the basis of their age, as well as their design features and design speed. For this study, four expressways of different designs were selected; although these four routes represent considerably different designs, none may be considered extreme in any sense.

- The first expressway, built specifically to standards for highways in the National System of Interstate and Defense Highways, is an Interstate route with a design speed of 70 miles per hour.
- The second expressway, a 15-year old parkway with a design speed of 50 miles per hour, was designed to standards that were considerably less rigorous in terms of both curvature and grade than presently are acceptable for Interstate highways in flat or rolling terrain.
- The third expressway, an intermediate highway in terms of both age and design criteria, is a 10-year old urban freeway having relatively modern curvature and grade characteristics and a design speed of 70 miles per hour. Its weakness lies in the substandard design of the acceleration and deceleration lanes.
- The fourth expressway, a highway having a geometric design comparable to that for the Interstate expressway with the exception of a higher magnitude of grade and curvature, had only partial control of access in the section used for this study; it had crossovers in the median and several at-grade intersections. In addition, substandard connections provided commercial establishments on the expressway with numerous points of access to a frontage road that followed the same route for most of the section under study.

In general, accomplishment of the basic aim of the study involved attempts to differentiate among these four different types of expressway designs; to examine the tension responses generated on these four expressways as functions of design characteristics and traffic interference; to determine the relation of tension responses to traffic volume; and to relate the results of the first two efforts to the design of other types of highways.

Procedure

Sections of the four test routes, each approximately 8½ miles long and generally close to the Washington, D. C., area, were chosen for this study. On two of the routes, the volume of traffic was relatively low, and they had no appreciable peak hours of traffic—less than 500 vehicles per hour in two lanes during daylight hours. Consequently, studies were made only during offpeak hours, from 10 a.m. to 3 p.m. On the other two routes, which are important expressways used for work trips into Washington, definite peak periods of traffic occurred. Test runs, timed to cover the periods of maximum traffic, were made on these two routes during morning and evening peak hours and also during the time corresponding to the offpeak hours on the other two routes. Prior to the beginning of this study, traffic volume counts were made on the routes with peak-hour traffic, during both the offpeak and peak hours, so that the GSR data collected could be related to traffic volume.

[1] Presented at the 41st annual meeting of the Highway Research Board, Washington, D.C., January 1962.

[2] References indicated by italic numbers in parentheses are listed on page 112.

Six test drivers were used; all were males and their ages ranged from 17 to 22 years. Two of the six had had previous experience in using the GSR equipment and were fairly familiar with the plan of the study and the operation of the instrument. Two teams of three drivers and a standard passenger car that had automatic transmission were used. Three people were in the test car during each run; and each member of this three-man team served on successive individual runs as a driver, an observer, and a data recorder.

The observer sat in the front seat with the driver and defined the cause of any change made in the position or speed of the test vehicle—interferences from traffic or from design characteristics of the highway—for the data recorder who entered the information on the GSR record. Factors considered as possible causes for changes in vehicle speed or position had been coded into eight categories, four were traffic related and four were design related. A list of these interferences is shown in table 1: numbers 4 through 7 apply to those attributed to highway characteristics, and the rest of the numbers apply to those attributed to traffic.

For each run, electrodes were fixed to the first and third fingers of the driver's left hand, and the sensitivity level of the GSR equipment was adjusted to a point at which a shock stimulus presented by the observer would cause a full-scale deflection of the recorder pen. Once adjusted, the sensitivity level was not changed while the particular driver was making his runs. Each driver covered the test route in one direction, took a short break, and returned. The travel times for the 8½-mile test sections varied from 8 to 21 minutes. Each of the six test drivers covered each of the four routes, as follows: 12 times each for offpeak and peak traffic hours on each of two routes, and 12 times each for each of the two routes that had no peak-hour traffic.

All data were recorded on chart paper; they included pertinent information about the route and driver as well as the GSR data. Because only the galvanic skin responses aroused by the specific, observable interferences were considered in this study, only the GSR data that were associated with the interferences listed in table 1 were analyzed. The basic measure of tension was defined as the magnitude of GSR per unit of time; this measure equalized the data for differences either in length of routes or in running times and tended to make the distribution of the GSR data more symmetrical than would have been obtained with GSR magnitude as the measure.

Tension Responses and Traffic Volumes

The relationship between tension responses and volume of traffic, which varied on the four routes from approximately 300 to 3,500 vehicles per hour in the two lanes, was of fundamental interest. The data collected for all routes were combined according to volume, and the curve of tension responses versus the traffic volume is shown in figure 1. Because this curve shows only the effect of traffic interferences on tension, it illustrates the direct relationship between driver tension and volume of traffic. The relationship seemed to be quite linear up to about 2,400 vehicles per hour in two lanes, and then the rise in tension appeared to increase exponentially. Tension-traffic volume data also were analyzed for the individual drivers and the same general form of the curve was found for all.

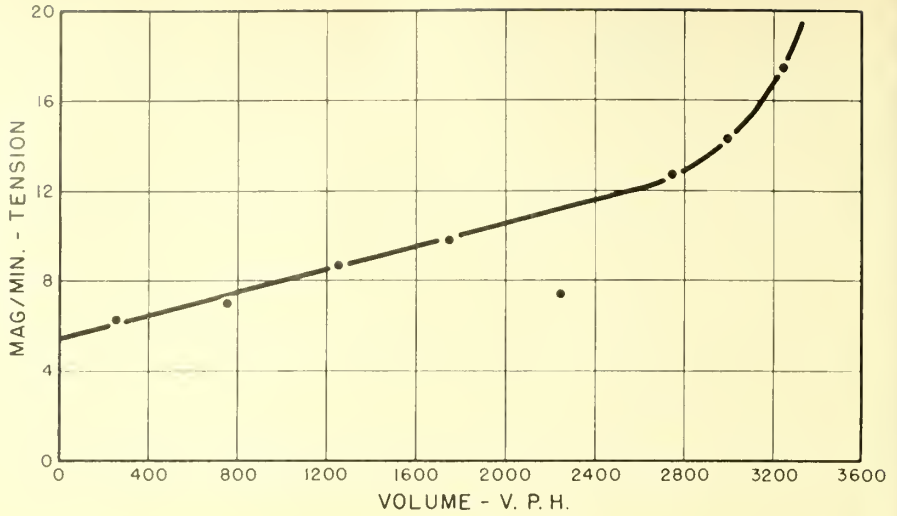

Figure 1.—Effect of traffic volume on tension responses.

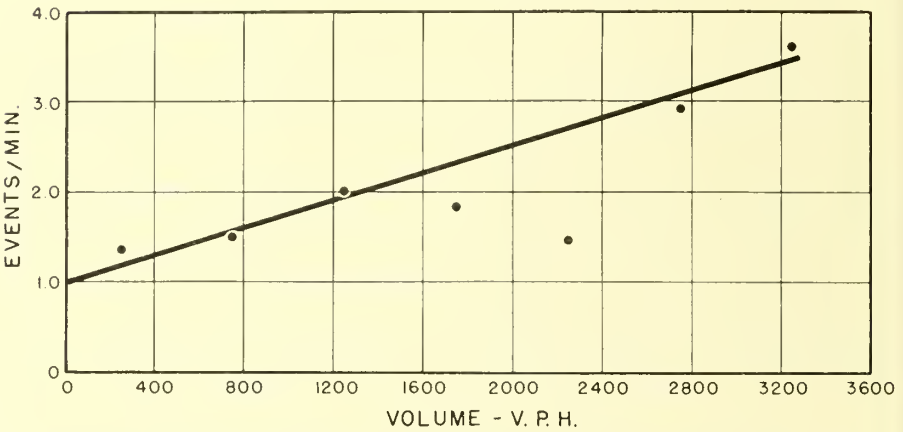

Figure 2.—Effect of traffic volume on rate of occurrence of interferences.

A two-way analysis of variance was performed on the data collected for driver tension responses to traffic volume, and an analysis was made of the trend of tension in relation to volume. The summary for these analyses is shown in table 2. The interaction term was found to be insignificant and was pooled with the residual. The results indicated a significant difference both among drivers and traffic volumes at better than the 0.01 level. In addition, the quadratic as well as the linear component of trend was significant at the 0.01 level. Thus, the form of the curve shown in figure 1 appears to be reliable.

A basic question in the use of the GSR concerned whether it was measuring something more than simply the frequency of occurrence of the interferences. If the same function defined the relation between interference per unit of time and the traffic volume as it did for driver tension and traffic volume, then the same results could be obtained simply by counting the number changes in the speed position of the vehicle. To examine this possibility, the number of traffic interferences per unit of time as a function of traffic volume was calculated, and the resultant data are plotted in figure 2. The same type of analysis of variance performed for the tension-traffic volume data was carried out on the interferences-per-minute data. The summary of this analysis is shown in table 3. In this analysis, as in the previous one, differences among the two major variables were significant. The linear trend among the volumes also was significant at the 0.01 level, but the quadratic component did not reach significance at this level. Thus the straight line relation shown in figure 2 was the best fit to the data.

From the two analyses of variance, it seems reasonable to conclude that the traffic interferences do induce a greater behavioral re-

Table 1.—Driving interferences

Interferences		
Identification Number	Name	Description
1	Instream vehicles	Conflicts caused by vehicles traveling in same direction.
2	Merging or crossing vehicles	Position or speed change caused by vehicles converging on test car.
3	Exiting vehicles	Position or speed change caused by vehicles diverging from traffic stream.
4	Gradient	Change in speed or position caused by grade.
5	Curvature	Change in speed or position caused by curvature.
6	Pavement changes	Position or speed change caused by variations in highway surface.
7	Shoulder objects	Position or speed change caused by shoulder objects such as cars or abutments.
8	Pedestrians	Changes caused by conflicts with pedestrians or animals.

Table 2.—Summary of analysis of variance on tension caused by traffic volume

Source of variance	Sum of squares	*df*	Mean square	F, ratio
Between subjects	2,034.38	5	406.88	[1] 25.70
Between volume	2,865.04	5	573.01	[1] 36.20
Error	1,535.76	97	15.83	
TOTAL	6,435.18	107		
Linear trend	1,705.45	1		[1] 107.73
Quadratic trend	117.72	1		[1] 7.44

[1] Significant at the 0.01 level.

Table 3.—Summary of analysis of variance on frequency of interferences caused by volume

Source of variance	Sum of squares	*df*	Mean square	F, ratio
Between subjects	15.99	5	3.20	[1] 4.77
Between volume	79.87	5	15.97	[1] 19.72
Error	78.16	97	0.81	
TOTAL	174.02	107		
Linear trend	70.15	1	70.15	[1] 86.60
Quadratic trend	4.47	1	4.47	5.52

[1] Significant at the 0.01 level.

Table 4.—Summary of analysis of variance of tension responses caused by traffic interferences

Source of variation	Sum of squares	*df*	Mean square	F, ratio
Subjects	1,122.6	5	224.52	[1] 11.79
Routes	468.0	3	156.0	[1] 8.19
Direction	32.5	1	32.50	1.71
Routes and subjects	300.6	15	20.04	1.05
Direction and routes	118.5	3	39.50	2.07
Direction and subjects	64.7	5	12.94	
Error	4,858.8	255	19.05	
TOTAL	6,965.7	287		

[1] Significant at the 0.01 level.

Table 5.—Summary of analysis of variance of highway characteristics (1–7)

Source of variation	Sum of squares	*df*	Mean square	F, ratio
Subjects	2,416.4	5	483.3	[1] 26.6
Routes	1,592.5	3	530.8	[1] 29.2
Direction	95.4	1	95.4	5.2
Routes and subjects	1,564.6	15	104.3	[1] 5.7
Direction and routes	19.1	3	6.4	
Direction and subjects	243.1	5	48.6	2.7
Error	4,633.2	255	18.2	
TOTAL	10,564.3	287		

[1] Significant at the 0.01 level.

sponse than is indicated simply by the frequency of their occurrences. Thus, the use of the GSR may be a behavioral measure of the operational efficiency of a highway, and also it may be a measure of the practical capacity of a highway.

Differentiating Among the Highways

The average magnitude of response per minute was determined for each test driver, for each route, and for the four traffic interferences during the offpeak hours. These data were subjected to an analysis of variance for which the summary is shown in table 4. No significant differences were noted between the data for directions, inbound vs. outbound, but significant differences were noted between data for the drivers and the data for the four routes.

Ordering the tension data according to highway, the highway built to Interstate standards generated less tension for each of the six drivers than the other three highways. Because this ranking included differences in tension caused by traffic volume, a correction was applied to the data shown in figure 1 to eliminate the effect of differences in volume—even during the offpeak hours, the urban freeway always carried three to four times more traffic than the other routes. All tension responses were corrected by multiplying them by a weight, which was the ratio of tension at a volume of 500 vehicles per hour to the tension at 1,250 vehicles per hour. An analysis of variance was performed with the corrected data and, as before, a significant difference among the routes was found. Now, the ranking of the four routes was still reliable, but the lowest level of tension was for the urban freeway, and the next higher levels of tensions were successively for the Interstate route, the parkway, and the freeway having only partial control of access.

The data for average magnitude of response per minute for offpeak traffic hours also were analyzed to determine the effects of interferences caused by highway design characteristics. Analysis of variance was performed in the same manner as for the traffic interferences. The results, which showed significant effects among the drivers and the routes, are given in table 5. Significant rank order among the highways also was determined; this order, from the lowest to highest tension induction was: the urban freeway, the parkway, the freeway having partial control of access, and the Interstate route.

GSR Magnitude Related to Interferences

An analysis for the average magnitude of GSR among the eight driving interferences was made. A rank test was employed rather than an analysis of the average magnitudes of the GSR themselves. Although a rank test is weak, its use avoids the necessity for meeting the distributional assumptions that would be required for stronger normal tests. The ranks for each route were compared; the test drivers

were considered as replicates. A summary for the four routes, with the significance of the rank order, is shown in table 6. This data shows the Interstate route had a ranking among the events that was significant at the 0.01 level. A comparison was made on the combined rankings of the four routes and the ranking of events was significant at better than the 0.01 level.

The ordering among the eight different interferences indicates very clearly that the traffic interferences consistently generated the highest magnitude of GSR. The highest average magnitude was generated by merging vehicles and the second highest by both instream conflicts and exiting vehicles. Among the highway characteristics, the highest magnitude of driver tension was induced by changes in pavement characteristics; this was followed very closely by that induced during negotiation of curves.

Frequency of Interferences

The importance of the rankings of the average magnitude of the GSR is meaningful, in part, according to the frequency with which the interferences actually occurred. Futher analysis of the distribution of the occurrence of the interferences was carried out on the data for all test drivers combined; the distributions for each of the highways are shown in table 7. Two interferences accounted for approximately 70 percent of them on all the routes: Interference No. 1, instream traffic interferences; and interference No. 5, negotiation of curves. The differences shown in table 7 indicate that on the urban freeway, instream interferences were considerably greater than the interferences of changes in curvature; this was expected because of the relatively high volume of traffic on this expressway even during the offpeak traffic hours. On the parkway, however, this pattern was reversed, which indicated the greater frequency and higher degree of curvature of this type of highway design. It is interesting to note that this reversal occurred even though the traffic volume was greater on the parkway than on the Interstate route. This reversal, therefore, indicated that the differences in GSR were caused by design characteristics.

Two groups of interferences

The eight interferences were divided into two groups for analysis; one for those caused by traffic and one for those caused by highway design characteristics. The frequency of occurrence for these interferences is given in table 8. Inspection showed considerable similarity of interferences among the four routes, the major difference being noted for those occurring on the parkway. This difference was indicated by the sharp increase in the number of interferences caused by highway curvature as opposed to the number of interferences from this source for the other three routes.

The data on the distribution of traffic interferences, also shown in table 8, indicate that instream conflicts are the dominant type of interference for drivers on freeways. These data were consistent for all routes for offpeak traffic hours between 90 and 95 percent of all interferences were instream conflicts. Information in table 8 also shows that on the high-volume urban freeway more than half of all the observed interferences were caused by traffic, but only approximately one-fourth of the interferences noted for the Interstate highway were caused by traffic.

Table 6.—Rank order of average magnitude of GSR generated by interferences for each route [1]

Rank [2]	Interstate highway	Urban freeway	Parkway	Expressway with partial control of access	All
1	2	2	2	3	2, merging vehicles
2	1	3	3	1	1, instream vehicles
3	7	1	1	2	3, exiting vehicles
4	6	5	6	8	6, pavement
5	5	6	7	6	7, shoulder objects
6	3	8	5	5	5, curvature
7	4	4	4	4	4, grade
8	8	7	8	7	8, pedestrians
	Reliability of rank $P<0.01$	Reliability of rank $P<0.15$	Reliability of rank $P<0.07$	Reliability of rank $P<0.11$	Reliability of rank $P<0.01$

[1] For definition of events see table 1.
[2] Ordering is from highest GSR average to lowest.

Table 7.—Percentage distribution of interference—offpeak data

Route	Interferences							
	1	2	3	4	5	6	7	8
Interstate highway:								
In	21.0	0.8	0.5	23.9	37.4	11.9	2.3	0.2
Out	24.0	0.7	0.3	26.8	39.1	3.9	1.8	0.2
Urban freeway:								
In	48.4	2.4	0.4	11.2	26.3	5.5	0.2	0.1
Out	51.9	1.2	0.9	12.9	22.3	5.2	0.8	0.4
Parkway:								
In	29.4	1.7	0.8	8.0	43.7	6.2	0.5	0.8
Out	28.3	1.1	1.4	11.3	41.9	6.2	0.7	0.5
Expressway with partial control of access:								
In	30.7	2.6	0.9	17.2	37.2	9.0	1.0	0.6
Out	31.3	2.2	1.0	18.0	38.8	5.5	0.8	0.2

Tension Induction on Freeway and Urban Arterial

Data also were available for two of the six test drivers for the same urban arterial studied previously (*1*), a four-lane rural primary highway with no control of access, and a freeway. These data, however, were restricted to traffic interferences and did not reflect tension caused by highway design characteristics. A comparison of data from these two highways and the high-type expressway is presented in table 9. The ratios of driver tension are shown in the last column of the table. The results of the comparison indicate the superiority of controlled-access design in reducing traffic interferences.

Discussion

The results of this study indicate that the GSR can be used as a measure to differentiate among types of expressway design. Although actual differences in the designs of the four expressways, all in good condition, were relatively small, significant differences among them were noted in terms of tension responses. The differences attributed to each of the two types of interferences studied demonstrate the effect of the different highway designs.

For traffic interferences, the urban freeway and the Interstate route were significantly less tension inducing than the other two highways. Actually, for the through driver, both of these roads were nearly comparable in terms of tension induction because the urban freeway has geometric design characteristics that meet Interstate standards over most of the study section. Marginal characteristics related to shoulders and ramps represent the deficiencies of the urban freeway, but when equated for traffic volumes, the two routes are very similar.

This study showed that, as far as the frequency and magnitude of traffic conflicts are concerned, highways designed to modern freeway standards are clearly superior to those more loosely designed. Control of access on expressways eliminates much of the marginal conflict for the through driver; this was demonstrated in the contrast between the Interstate route and the design having only partial control of access. The latter was consistently the most tension inducing route; the major difference was in an increase in the frequency of the occurrence of conflicts with merging and exiting vehicles, that is, marginal interferences. This difference was further shown in the comparisons of the GSR data for the primary and urban arterial. These routes generated around 30 percent of their conflicts from marginal interferences, but the high-type expressway generated less than 10 percent from such interferences.

Table 8.—Percentage distribution of highway and traffic interferences—offpeak hours

Route	Highway interferences				Traffic interferences				Percentage of total interferences from traffic
	4	5	6	7	1	2	3	8	
Interstate highway:									
In	31.7	49.5	15.8	3.0	93.0	3.8	2.3	0.9	23.0
Out	37.4	54.5	5.5	2.6	95.5	2.7	1.1	0.7	26.0
Urban freeway:									
In	25.9	60.9	12.7	0.5	94.3	4.7	0.9	0.1	54.4
Out	31.2	54.2	12.6	1.9	95.4	2.2	1.7	0.7	56.9
Parkway:									
In	13.8	74.9	10.5	0.8	89.8	5.2	2.4	2.6	35.9
Out	18.8	69.7	10.3	1.1	90.5	3.6	4.5	1.4	34.2
Expressway with partial control of access:									
In	26.7	57.7	14.0	1.6	88.4	7.4	2.5	1.7	35.1
Out	28.5	61.5	8.7	1.2	90.2	6.3	2.9	0.7	35.5

Table 9.—Tension generated on three types of highways

Type of highway	Tension, magnitude/minute			Ratio of tension on three routes to tension on expressway		
	Driver A	Driver B	Average	Driver A	Driver B	Average
Controlled access	5.7	5.5	5.6	1.00	1.00	1.00
Primary with uncontrolled access	10.8	8.8	9.8	1.89	1.60	1.75
Urban arterial	13.9	23.5	18.7	2.44	4.27	3.34

A similar but more subtle interaction was oted for the parkway; the tolerance of high urvature and gradient interacted with the raffic interferences to increase the level of ension for the drivers. The driver had ncreased difficulties in handling the conflicts n traffic when he also had to cope with rather arge changes in the geometries of the highway self.

The tension-producing relationships among he highways lend support to the hypothesis, roposed in the previous study with GSR (*1*), hat one of the basic determinants of driver ension is the degree of predictability that xists in the driving environment. It was bvious from this study that, under high-olume traffic conditions, the driver is inter-cting with vehicles around him and must ondition his performance to his expectation f what other vehicles are doing and will do. n general, he does not have enough informa-ion to develop stable or reliable predictions bout the activities of these other vehicles.)n a highway having only partial control of ccess, his problem is confounded by the ncrease in marginal activity, especially when oth entering and exiting interferences involve arge, angular closing rates. Thus, increasing raffic volume, increasing marginal activity, nd increasing variations in the highway itself ll contribute to the complexity of the driving nd in turn make it more difficult for the river to develop stable predictions about is driving environment.

Highway rankings

The results of the rankings of the routes for he highway characteristics are rather nomalous. The Interstate route, which perated well relative to traffic interferences, generated the highest tension from highway nterferences. The resolution of this paradox nay well be the differences in travel speed on hese highways. A systematic difference among the four expressways occurred in terms of the speed adopted by the drivers—an average of: between 60 and 65 miles per hour on the Interstate route, nearly 50 miles per hour on the urban freeway, and nearly 40 miles per hour on the parkway.

The increasing speeds indicated that drivers compensate for infrequent traffic interferences—either from low volume of traffic or good highway design—by traveling faster. In other words, drivers tend to make their speeds contingent upon the perceived complexity of the driving situation. In effect, the design of the Interstate route permitted a driver to increase his speed to the point at which the highway characteristics of curvature, grade, and pavement condition began to affect his operation of the vehicle. Such a conclusion would suggest that drivers adopt some kind of a critical level of driving tension.

In these terms, tension induced in driving may well represent one mechanism by which the driver can stabilize the system. That is, by driving at or near the speed at which tension responses increase sharply, the driver will be able to determine qualitatively an upper limit to his control over the driving situation. Obviously, this kind of criterion will be applicable to interferences caused by either traffic or highway conditions, or both. When traffic conditions are such that the driver is subject to considerable stress, he will reduce his speed and thereby decrease the frequency of tension-inducing stimuli. When traffic is not a factor, he will utilize the highway characteristics and drive sufficiently fast to get information from the road itself to give him a measure of performance.

Comfort and convenience

The results of this study also bear on the problem of comfort and convenience. For many years, it has been known that driver choices among alternative routes could not be accounted for either on the basis of economy of operation or of time. It has been necessary, therefore, to postulate the additional factor of comfort and convenience. The basic problem with such a construct is to develop an operational definition that will make it measurable. Differences in tension responses on different highways may represent one avenue for resolving this problem.

Data in this study indicate that a rural primary highway as an alternate route for a freeway generates twice as much tension as the freeway itself. Furthermore, on the highways such as this that have no control of access, nearly 30 percent of the traffic interferences arose from marginal conflicts, but on the freeway less than 10 percent of the traffic interferences arose from these sources. However, little difference was noted in tension generated by instream conflicts, except that fewer of these conflicts occurred on the freeway. Thus, two major factors appear to account for the differences in tension generated by the freeway and the primary having uncontrolled access: (1) proportion of marginal interferences, and (2) frequency of instream conflict. Such a breakdown suggests a logical distinction between comfort and convenience. Thus, the comfort of a route may be defined as the tension caused by unpredictable conflicts. Considered in terms of the predictability of the interferences, route comfort appears to be measurable by use of the GSR.

Convenience may be defined as the degree of freedom that a driver has in setting the level of performance of his own system. Elements in the route that restrict the driver or force conformity to external controls would make that route inconvenient. For example, a wide variety of traffic control devices generally are predictable, but they force the driver to make control changes that may conflict both with the operation of his system and his driving objectives. Similarly, interaction with other vehicles in the traffic stream frequently is predictable, at least at moderate volumes of traffic, yet it restricts the driver's freedom of action. In this respect, it is interesting to note that the relation between tension and traffic volume, shown by data in figure 1, breaks sharply around 2,800 vehicles per hour or an average of 1,400 vehicles per lane per hour. This may represent the point at which the traffic situation becomes highly unpredictable; in terms of this discussion, the point at which driving would change from being inconvenient to being uncomfortable. Because the data of this study show only small differences in the average GSR from instream interferences, it is entirely possible that their frequency of occurrence alone may be an adequate measure of convenience. Claffey (*2*) used such a measure in his studies of comfort and convenience, but he made no distinction between the two factors.

It is difficult to determine the weighting of the two factors of marginal and instream conflicts to fit some route choice equation. However, by using the GSR as an overall measure of both factors, the data given in table 9 show that the freeway generated the least tension; the primary route having un-

controlled [illegible] generated 1.75 times more tension and the arterial highway generated 3.34 times more tension than the freeway. By subjective responses, the drivers evaluated the three routes in a direct but non-linear relation with tension; that is, their dislike of a route increased more rapidly than tension increased. Considerable research will be required to verify this relation and factors related to the choice among alternative routes.

Comparison of the data from this study clearly shows the superiority of modern expressway design over other types of highway design. Nearly all traffic interferences were minimized by these modern designs, except for certain of those occurring within the traffic stream. Thus, even under high-volume traffic conditions, modern freeway design will help to restrict the type of conflicts with which a driver must deal to those that are the easiest for him to resolve efficiently. However, this study also indicated that modifications in highway design alone may not necessarily increase overall system stability.

REFERENCES

(1) *Tension Responses of Drivers General on Urban Streets*, by R. M. Michaels, Highw Research Board Bulletin 271, 1960, pp. 29–4

(2) *Characteristics of Passenger-Car Tra on Toll Roads and Comparable Free Roads j Highway User Benefit Studies*, by P. J. Claff Public Roads, vol. 31, No. 8, June 1961, p 167–176.

PAVEMENT RESEARCH

(Second AASHO Road Test Film)

The second film produced in connection with the AASHO Road Test, *Pavement Research*, has been released by the Bureau of Public Roads. This 16-mm. color film has a running time of 37 minutes; it shows the tests made on rigid and flexible-type pavements, the rationale for analysis of the data, and the principal test results. As a companion film to *Materials and Construction*, which recorded the Test for the 1956–1958 period, *Pavement Research* summarizes the program for the 1958–1961 period. A short description of the AASHO Road Test and production of these two films appears in PUBLIC ROADS, vol. 32, No. 3, August 1962, p. 63.

Prints of *Pavement Research*, and of the first film *Materials and Construction*, are available on a loan basis from the Bureau of Public Roads, Photographic Section, 1717 H Street NW., Washington 25, D.C. The prints may be borrowed by any responsib organization. There is no charge other tha for express or postage fees. Requests shou be submitted well in advance of the desir showing date, and alternate dates should indicated, if possible. Immediate return required. Inquiries about purchase of t film or films should be addressed to t Public Roads Photographic Section.

Passenger Car Fuel-Consumption Rates

BY THE ECONOMIC RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

By NATHAN LIEDER,
Statistician

Information on fuel-consumption rates of passenger cars presented in this article was collected primarily to provide data for the report submitted to Congress as part of the Bureau of Public Roads Highway Cost Allocation Study. The analysis in the article is more detailed than could be prepared for that report.

Fuel-consumption rates have many uses, the paramount one being in the forecasting of tax revenues that will be available for highway programs. Fuel-consumption rates also are helpful tools for measuring the use made of highways and for determining the fairness of the tax burdens imposed on different types of vehicles.

The findings on passenger-car fuel-consumption rates are expected to be useful to highway administrators and planners and to others requiring information on fuel-consumption rates. These findings reflect the actual, normal daily use of many privately-owned passenger cars rather than reports for test vehicles or for those employed for a few specialized purposes.

Introduction

FUEL-CONSUMPTION rates enter into estimates for fuel tax contributions of the different classes of motor-vehicles. Estimates of the tax yield have many uses; two may be noted: (1) determination of highway-user tax schedules that rest equitably on the classes of vehicles, and (2) calculation of benefit-cost analyses for highway system segments for which the traffic composition can be postulated. To obtain the best possible measures, past bases for estimating rates of fuel-consumption must be examined, and they must be revised, if need be, to reflect more accurately the changes in vehicles and their use.

The Bureau of Public Roads estimated average motor-vehicle payments to the Highway Trust Fund as part of the Highway Cost Allocation Study required by Section 210 of the Highway Revenue Act of 1956 (*1*).[1] Estimated fuel-consumption rates served as one basis for calculating the average vehicle payments. The rate for passenger cars was based, in part, upon data on the use of privately-owned passenger cars that had been submitted for specified periods between October 1959 and March 1961 by groups of employees of nine State highway departments and the corresponding Division, and some Regional, offices of the Bureau of Public Roads. Because all reports were not available in time to permit a detailed analysis for use in the report to Congress, this article presents an analysis of all the data collected. The findings should be of use to administrators and planners because they reflect the actual, daily use of many privately-owned passengers cars rather than reports for test vehicles or for those employed for a few specialized uses.

[1] References indicated by italic numbers in parentheses are listed on page 120.

Summary

The major findings of this investigation of the rates of motor-fuel consumption reported for a number of privately-owned passenger cars in normal daily use are, as follows.

- Cars in class 0, those having six cylinders including compacts, consumed less gasoline in daily operation than the standard American cars used in the study.
- For any specified vehicle-transmission class, a change of ten percent in mileage driven at speeds of 35 miles per hours or less caused a corresponding change of 0.002 gallon per mile in the fuel-consumption rate—either increase or decrease.
- Year model of the vehicle did not affect the average fuel-consumption rates sufficiently to serve as an efficient factor for use in forecasting gas consumption.

Procedure

As shown in table 1, the nine States participating in this study began the collection of data at different times during the period October 1959 to March 1960, and each State collected reports for four seasons. Most of the States used data from a different group of employees each season; but, Connecticut and Illinois used the data from one slate of employees for all four seasons; and New Mexico used data from two groups, one for the first two seasons and the other for the remaining two seasons.

Each participating employee was given a form on which to record information concerning the vehicle, the mileage driven, and the amount of fuel consumed. Acceptable forms had to contain a record of four or more purchases of gasoline and the first and last purchases had to show a full gas tank. An early edition of the model form distributed for this study contained neither space requiring the reporting of the number of engine cylinders for each car nor the date of each gas purchase. Consequently, California and Arizona did not collect information on the number of cylinders, and most of the States did not ask participants to record the date of each fuel purchase. Any follow-up study should require the reporting of information for these two items.

With the exception of Utah, the States that selected more than one group of employees for the study experienced less seasonal variation in the number of reports received than Connecticut and Illinois, where only one group of employees had been used. The fact that employee participation was completely voluntary, coupled with the possibility that the enthusiasm of the employees waned with time, may have been a factor in the seasonal variation in the number of responses. Change in employment of some employees is thought to have been another factor contributing to the differences in the number of responses from season to season. It may be hypothesized that the first factor was a very influential cause of the variation in seasonal participation in the States that used a single employee group. But, even a one-hundred-percent participation in all States would not guarantee that the reports so faithfully mirrored the vehicle population of a State or the Nation as to permit the use of unweighted data in the estimation of average fuel-consumption rates for all motor-vehicles. Therefore, this article reports unweighted averages and users may supply appropriate weights to suit different situations. However, a set of national averages obtained by special weighting of the reported data is presented in table 10.

Factors Studied

The relationship of fuel-consumption rates to factors of vehicle weight, engine size (horsepower), and transmission type of vehicle, and to season of the year and to stop-and-go driving have been analyzed in earlier studies. The analysis presented here was based upon these factors or related factors, plus vehicle year model. Planners and administrators can find State or national data for such factors in the records of highway

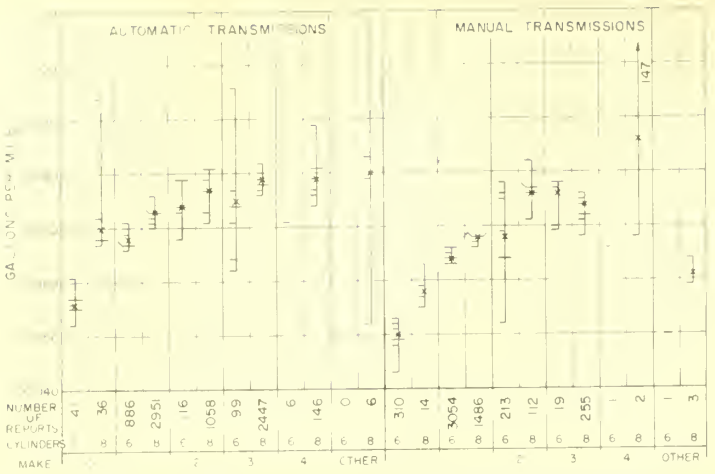

Figure 1.—Average fuel-consumption rates for vehicles for each reporting State and total number of reports by vehicle make class, transmission type, and number of cylinders: 1960.

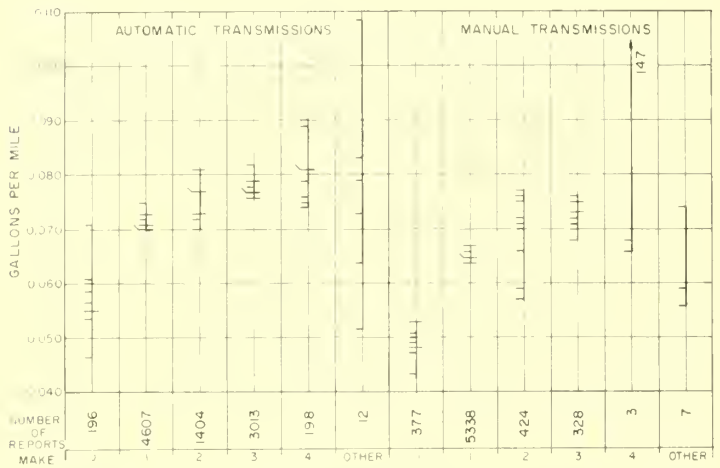

Figure 2.—Average fuel-consumption rates for vehicles for each reporting State and total number of reports by vehicle make class and transmission type: 1960.

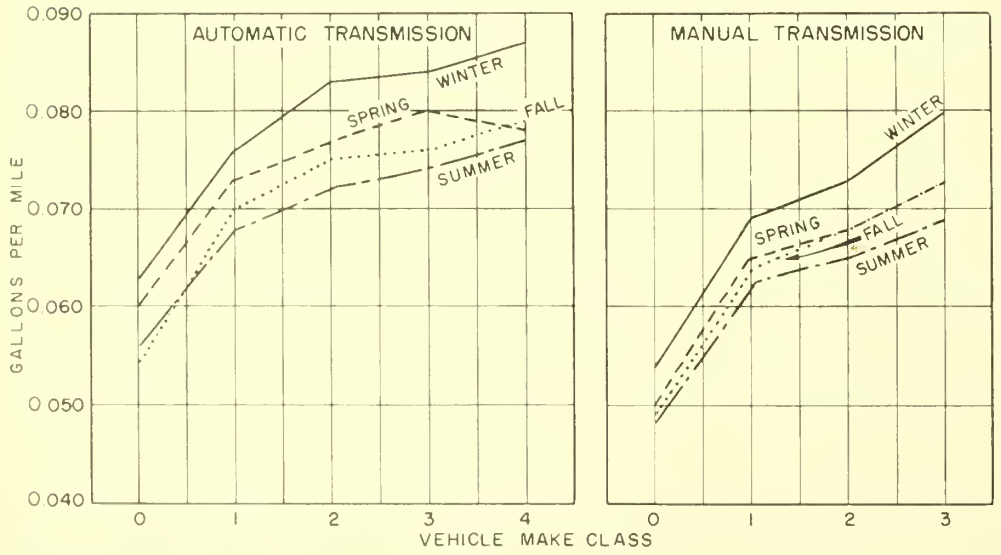

Figure 3.—Average fuel-consumption rates by season for vehicles in each make class by transmission type: 1960.

departments, motor-vehicle administrator automobile manufacturers, and simila sources. Because other analysts concerne with the tax yield from fuel consumption hav not found it feasible to collect data for drive habits, quality of vehicle maintenance, an octane rating of gasoline used, no attemp was made to include such factors in this study

Because it was impracticable to weigh th vehicles for which reports were received, n quantitative relationship between weigh and fuel-consumption rates could be estab lished in this investigation. To take weigh into account in the analysis, standard Amer can cars were grouped by make into fiv classes roughly indicative of weight. Som cars that could not be included in any o the other five classes have been grouped i one category, "other." Two consideration were used in making car assignments to particular class: the number of vehicle registered for each year model of the make and the estimated empty weight of the four door sedan judged to be the most popular fo each year model. The assignments of th makes of cars to each class are shown in tabl 2. It is recognized that wide differences i weight may exist within a single make. How ever, a relatively inexact measure based upo obtainable data should prove acceptable fo broad-scale planning, provided it does no mask significant differences in fuel-consump tion rates. In this investigation, the roug measures of weight by vehicle class did no seem to obscure marked differences in fue consumption rates. Few vehicles of foreig make were included in the study; four State did not report the make of foreign vehicle and; therefore, those included were no classified by make.

The number of cylinders for each car wa used as a rough measure of its engine siz (horsepower). The strength of the relation ship was not studied, but it is believed t have been sufficient for the purposes to b served by the analysis. Moreover, th number of cylinders could be reporte objectively by all participants, whereas engin size could not.

Data for the factor of stop-and-go drivin were based on the memory and judgment o the participants. At the time of each fue purchase, when the number of gallons o gas purchased and the odometer reading wer being recorded, each participant was aske also to record his estimates of either th percentage of mileage or the number of mile that had been driven at speeds of 35 mile per hour or less since the previous gas pur chase. A weighted total of this mileag was calculated from each participant's report Although speeds of 35 miles per hour or les are not always associated with the stop-and go driving experienced on urban streets they should be indicative of such driving Provided that any bias caused by failur in memory or poor judgment of the respon dents was small, differences in fuel-consump tion rates should be correlated with th differences in the proportions of stop-and-g urban driving.

ANALYSIS

The States sent either a set of duplicate cards or a listing of coded responses to the Washington, D.C., office of the Bureau of Public Roads. Because a few cards and coded listings were rejected, the totals shown in tables and figures have minor differences from the information transmitted. The following analysis was based upon the resultant deck.

Make Class, Transmission Type, and Cylinders

Average fuel-consumption rates for American cars listed by States submitting reports as to: make class, transmission type, and number of cylinders are shown in tables 3, 4, and 5 and in figures 1 and 2. California and Arizona are not represented in figure 1 because responses from these States did not list the number of cylinders for the cars. The number of observations is the total of all acceptable seasonal reports. For some States, these observations represent the same vehicles for all four seasons; for other States, the observations represent different sets of vehicles and probably a different set of drivers each season. The vertical lines in figures 1 and 2 depict the range of fuel-consumption rates averaged for each State. Each short, horizontal line perpendicular to a vertical line represents the average fuel-consumption rate for the vehicle class for a State. Two States having the same average are represented by short horizontals on either side of the vertical. Each additional State having the same average as two other States is represented by a short appendage to the horizontal. The "x" in each vertical represents the average fuel-consumption rate found in the study for all vehicles of a given classification. The study averages are not necessarily national averages.

Because Illinois collected almost half of the observations, its reports weight the study averages more than those of any other State. Parenthetically, it may be noted that, with one exception, the average fuel-consumption rates found in the Illinois Study are not at either extreme of any of the distributions of State averages. The one exception is in the class of other American cars having eight cylinders and automatic transmissions. The Illinois average for this class is based on two observations.

Make class of cars

Several deductions may be drawn from the data shown in tables 3 and 4, and figure 1. Probably the most significant one concerns vehicles in class 0, which contained American compact cars. The average fuel-consumption rates for the cars in class 0 were smaller than the average rates for the other classes of American cars except for the eight-cylinder cars in class 0 that had automatic transmissions. The contrast was more definitely established for the six-cylinder cars than for the eight-cylinder cars in class 0. The low average rate of fuel-consumption for the eight-cylinder cars in class 0 is probably an

Table 1.—Participating States in each Region, starting period, and number of records tabulated

Public Roads Region, and State	Starting period	Number of records tabulated [1]				
		Spring	Summer	Autumn	Winter	Total
Region 1:						
Connecticut	January 1960	85	93	76	149	403
Region 3: [1]						
North Carolina	January 1960	190	178	182	207	757
Region 4:						
Illinois	January 1960	2,079	1,909	1,688	2,265	7,941
Region 5:						
Kansas	October 1959	196	182	205	204	787
Region 7:						
Arizona	January 1960	184	184	208	204	780
California	January 1960	523	502	481	470	1,976
Region 8:						
Oregon	March 1960	251	219	208	208	886
Region 9: [1]						
New Mexico	January 1960	447	409	450	412	1,718
Utah	February 1960	341	304	265	277	1,187
Total		4,296	3,980	3,763	4,396	16,435

[1] Includes reports submitted by employees of the Regional office of the Bureau of Public Roads, which were not tabulated by the States.

Table 2.—American-make cars grouped in classes, roughly indicative of weight

Class 0	Class 1	Class 2	Class 3	Class 4	Other
Corvair	Chevrolet	Dodge	Buick	Cadillac	Corvette
Crosley	Ford	Hudson	Chrysler	Continental	Hawk
Falcon	Plymouth	Kaiser-Frazer	DeSoto	Imperial	Jeep
Henry J	Studebaker	Nash	Edsel	Lincoln	Thunderbird
Lark		Pontiac	Mercury		
Rambler			Oldsmobile		
Valiant			Packard		
Willys			LaSalle		

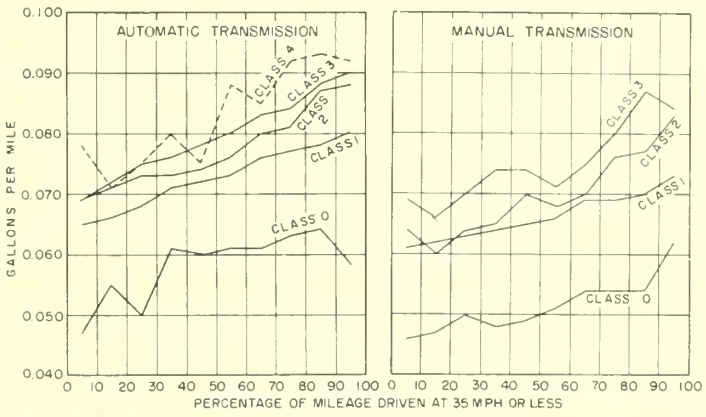

Figure 1.—Average fuel-consumption rates of American cars by percentage of miles driven at 35 m.p.h. or less for vehicles in each make and transmission class: 1960.

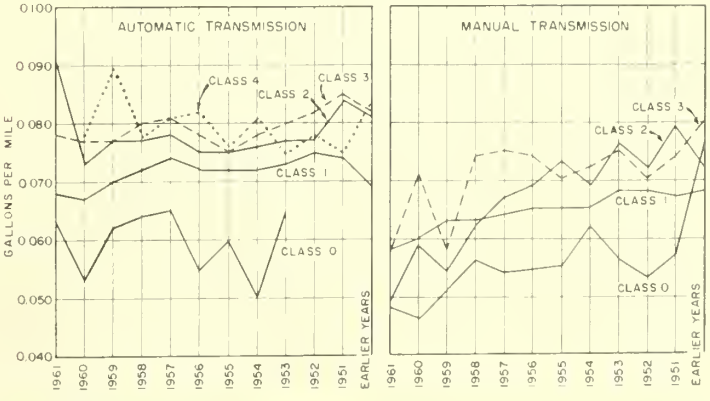

Figure 5.—Average fuel-consumption rates by year model for vehicles in each make and transmission class: 1960.

Table 3.—Data on miles of travel and fuel-consumption rates, gallons per mile, for American cars with automatic transmissions, taken from 1960 reports that listed number of cylinders [1]

Location and vehicle make class	Automatic transmission								
	6 cylinders			8 cylinders			Total		
	Reports	Travel	Fuel	Reports	Travel	Fuel	Reports	Travel	Fuel
	Number	*Veh.-mi.*	*Gal./miles*	*Number*	*Veh.-mi.*	*Gal./miles*	*Number*	*Veh.-mi.*	*Gal./miles*
Connecticut: [2]									
Class 0	2	2,937	0.055	0	0	----	2	2,937	0.055
Class 1	29	34,076	.067	86	109,989	0.071	115	144,065	.070
Class 2	3	3,572	.068	22	29,060	.071	25	32,632	.070
Class 3	11	11,789	.074	65	76,970	.080	76	88,759	.079
Class 4	0	0	----	4	3,882	.089	4	3,882	.089
Other	0	0	----	0	0	----	0	0	----
All	45	52,374	.068	177	219,901	.074	222	272,275	.073
North Carolina: [3]									
Class 0	7	12,609	.055	0	0	----	7	12,609	.055
Class 1	50	53,189	.067	196	245,164	.070	246	298,353	.070
Class 2	8	6,631	.074	59	62,855	.073	67	69,486	.073
Class 3	2	2,128	.062	115	131,953	.076	117	134,081	.076
Class 4	0	0	----	8	10,828	.076	8	10,828	.076
Other	0	0	----	0	0	----	0	0	----
All	67	74,557	.065	378	450,800	.072	445	525,357	.071
Illinois: [3]									
Class 0	88	123,852	.056	15	18,511	.068	103	142,363	.057
Class 1	613	697,568	.067	1,672	1,817,931	.073	2,285	2,515,499	.072
Class 2	66	62,839	.074	597	658,998	.077	663	721,837	.077
Class 3	58	50,062	.077	1,530	1,665,695	.079	1,588	1,715,757	.079
Class 4	0	0	----	59	70,628	.079	59	70,628	.079
Other	0	0	----	2	1,331	.108	2	1,331	.108
All	825	934,321	.067	3,875	4,233,094	.076	4,700	5,167,415	.075
Kansas: [3]									
Class 0	5	5,829	.061	3	2,474	.094	8	8,303	.071
Class 1	55	58,998	.069	221	234,155	.076	276	293,153	.075
Class 2	4	4,388	.079	62	64,156	.081	66	68,544	.081
Class 3	2	1,853	.064	144	153,834	.082	146	155,687	.082
Class 4	0	0	----	10	12,914	.074	10	12,914	.074
Other	0	0	----	1	1,026	.083	1	1,026	.083
All	66	71,068	.069	441	468,559	.079	507	539,627	.078
Oregon: [2]									
Class 0	18	25,894	.057	3	4,235	.072	21	30,129	.059
Class 1	45	50,762	.066	178	206,248	.072	223	257,010	.071
Class 2	19	21,126	.073	91	98,016	.077	110	119,142	.077
Class 3	6	6,041	.074	118	138,998	.078	124	145,039	.077
Class 4	6	6,820	.071	18	20,494	.077	24	27,314	.075
Other	0	0	----	0	0	----	0	0	----
All	94	110,643	.066	408	467,991	.075	502	578,634	.073
New Mexico: [2]									
Class 0	12	15,248	.057	9	11,095	.067	21	26,343	.061
Class 1	56	61,274	.070	331	394,332	.073	387	455,606	.073
Class 2	7	7,221	.079	116	120,583	.081	123	127,804	.081
Class 3	6	3,945	.096	223	253,389	.078	229	257,334	.078
Class 4	0	0	----	30	35,741	.081	30	35,741	.081
Other	0	0	----	2	2,372	.079	2	2,372	.079
All	81	87,688	.070	711	817,512	.076	792	905,200	.075
Utah: [3]									
Class 0	8	10,782	.052	6	10,532	.068	14	21,314	.060
Class 1	38	40,613	.071	267	334,856	.073	305	375,469	.073
Class 2	9	10,715	.074	111	124,805	.079	120	135,520	.077
Class 3	14	15,087	.071	252	309,402	.077	266	324,489	.077
Class 4	0	0	----	17	18,915	.081	17	18,915	.081
Other	0	0	----	1	1,452	.052	1	1,452	.052
All	69	77,197	.069	654	799,962	.075	723	877,159	.075
All agencies:									
Class 0	140	197,151	.056	36	46,847	.070	176	243,998	.059
Class 1	886	996,480	.068	2,951	3,342,675	.073	3,837	4,339,155	.072
Class 2	116	116,492	.074	1,058	1,158,473	.077	1,174	1,274,965	.077
Class 3	99	90,905	.075	2,447	2,730,241	.079	2,546	2,821,146	.079
Class 4	6	6,820	071	146	173,402	.079	152	180,222	.079
Other	0	0	----	6	6,181	.080	6	6,181	.080
All	1,247	1,407,848	.067	6,644	7,457,819	.076	7,891	8,865,667	.075

[1] Nineteen observations for 4-cylinder Willys are excluded from the tabulations.
[2] Includes reports from employees of Division Offices of Bureau of Public Roads and State highway employees.
[3] Includes reports from employees of both the Division and Regional Offices of the Bureau of Public Roads and State highway employees.

unreliable estimate based upon too small a sample of the vehicles in this category. However, the differential in fuel-consumption rates points up the advantage of considering vehicle distributions by various characteristics that influence fuel-consumption before making estimates of the gallons of fuel to be consumed in highway use.

Transmission type

For all classes of cars, this study confirmed that the type of transmission and engine size as measured by number of cylinders have an effect on fuel-consumption rates. With the exception of class 0 vehicles, the effect of the type of transmission for cars in every day use seems to have been more marked than the effect of the number of cylinders. Therefore, computation of fuel-consumption rates in which the number of cylinders is not considered should yield estimates acceptable for many purposes. Furthermore, by ignoring the effect of the number of cylinders, the data received from California and Arizona could be included in the computations for a unified analysis. In figure 2, State average are shown but no differentiation by transmission type has been made, as in figure 1.

Differences in rates among States

Differences in average fuel-consumption rates among States for any given class of car were related to the number of observations and decreased with a large number of observations. This was demonstrated most markedly by the data received for class 1 vehicles. The decrease in the differences of average fuel consumption rates among the States studied indicated that the average rate of fuel-consumption in a vehicle weight class for any State approaches that of the other States without regard to their geographical location. It is possible that this similarity of fuel consumption rates would not be maintained in mountainous areas or at high altitudes as fuel-consumption rates have been directly related to changes in altitude. Although this study was not designed to provide information on the relative importance of this factor, reports of several other studies have shown that altitude does affect the rate of fuel consumption (*2*).

Miles Per Gallon

Because the data in this article are expected to be used for purposes requiring fuel-consumption rates to be weighted by miles of travel, the data have been expressed as gallons per mile. However, many readers of this article and perhaps many of the prospective users of the data are more familiar with rates expressed as miles per gallon. Table 6 therefore, contains fuel-consumption rates expressed as miles per gallon that correspond to the gallon-per-mile rates calculated from the reports received from all participants for each of the six classes of American cars, as shown in table 5.

Seasonal Variation

Data shown in table 7 and figure 3 confirmed that fuel-consumption rates vary in relation to the season of the year. Rates at either extreme were reported for summer and winter. The smallest rates were reported for summer and the largest rates were reported for the winter season. Spring and fall reports indicated intermediate rates of fuel-consumption, which generally were near the annual average. The rates given in table 7 may be used with forecasts of seasonal travel to produce somewhat more refined forecasts of total fuel to be consumed than can be produced without a consideration of the seasonal differences in consumption.

Stop-and-Go Driving

For each transmission type within each make class, the rate of fuel consumption tended to vary directly as the proportion of stop-and-go driving changed, as measured by the percentage of driving speeds reported as not exceeding 35 m.p.h. Data in table 8 and figure 4 illustrate this relationship. The jagged progression of data for class 0 and

Table 4.—Data on miles of travel and fuel-consumption rates, gallons per mile, for American cars with manual transmissions, taken from 1960 reports that listed number of cylinders [1]

Location and vehicle make class	Manual transmission								
	6 cylinders			8 cylinders			Total		
	Reports	Travel	Fuel	Reports	Travel	Fuel	Reports	Travel	Fuel
	Number	*Veh.-mi.*	*Gal./miles*	*Number*	*Veh.-mi.*	*Gal./miles*	*Number*	*Veh.-mi.*	*Gal./miles*
Connecticut: [2]									
Class 0	6	11,211	0.048	0	0	------	6	11,211	0.048
Class 1	88	112,325	.063	25	31,221	0.067	113	143,546	.064
Class 2	12	19,976	.052	4	4,329	.077	16	24,305	.057
Class 3	0	0	------	10	13,136	.068	10	13,136	.068
Class 4	0	0	------	0	0	------	0	0	-----
Other	0	0	------	0	0	------	0	0	-----
All	106	143,512	.060	39	48,686	.068	145	192,198	.062
North Carolina: [3]									
Class 0	13	19,479	.043	0	0	------	13	19,479	.043
Class 1	181	215,585	.064	116	131,667	.068	297	347,252	.065
Class 2	13	8,032	.076	6	6,790	.077	19	14,822	.076
Class 3	2	1,530	.069	28	31,644	.072	30	33,174	.072
Class 4	0	0	------	0	0	------	0	0	-----
Other	0	0	------	0	0	------	0	0	-----
All	209	244,626	.063	150	170,101	.069	359	414,727	.065
Illinois: [3]									
Class 0	159	232,473	.049	4	5,151	.063	163	237,624	.050
Class 1	1,819	2,030,607	.064	774	864,000	.068	2,593	2,894,607	.065
Class 2	124	126,410	.069	60	68,005	.076	184	194,415	.072
Class 3	7	4,838	.077	128	135,654	.076	135	140,492	.076
Class 4	0	0	------	0	0	------	0	0	-----
Other	1	1,767	.050	1	1,273	.064	2	3,040	.056
All	2,110	2,396,095	.063	967	1,074,083	.069	3,077	3,470,178	.065
Kansas: [3]									
Class 0	14	16,007	.053	1	1,481	.059	15	17,488	.053
Class 1	148	157,402	.066	77	86,648	.068	225	244,050	.067
Class 2	8	8,894	.064	6	5,149	.082	14	14,043	.070
Class 3	1	1,010	.078	7	9,111	.071	8	10,121	.071
Class 4	0	0	------	1	1,474	.068	1	1,474	.068
Other	0	0	------	0	0	------	0	0	-----
All	171	183,313	.065	92	103,863	.069	263	287,176	.066
Oregon: [2]									
Class 0	35	58,625	.049	0	0	------	35	58,625	.049
Class 1	186	226,029	.063	81	100,601	.066	267	326,630	.064
Class 2	10	10,209	.078	6	5,812	.077	16	16,021	.077
Class 3	2	2,483	.070	17	18,304	.074	19	20,787	.073
Class 4	1	1,132	.066	0	0	------	1	1,132	.066
Other	0	0	------	0	0	------	0	0	-----
All	234	298,478	.061	104	124,717	.067	338	423,195	.063
New Mexico: [2]									
Class 0	48	64,579	.052	5	6,625	.057	53	71,204	.053
Class 1	442	477,937	.066	278	323,252	.068	720	801,189	.067
Class 2	24	20,600	.075	19	22,960	.076	43	43,560	.075
Class 3	7	5,418	.078	41	48,411	.072	48	53,829	.073
Class 4	0	0	------	1	433	.147	1	433	.147
Other	0	0	------	2	2,099	.059	2	2,099	.059
All	521	568,534	.065	346	403,780	.069	867	972,314	.067
Utah: [3]									
Class 0	35	52,172	.051	4	9,880	.055	39	62,052	.051
Class 1	189	229,214	.065	134	160,690	.068	323	389,904	.066
Class 2	22	28,434	.064	11	13,382	.071	33	41,816	.066
Class 3	0	0	------	24	27,092	.075	24	27,092	.075
Class 4	0	0	------	0	0	------	0	0	-----
Other	0	0	------	0	0	------	0	0	-----
All	246	309,820	.062	173	211,044	.068	419	520,864	.065
All agencies:									
Class 0	310	454,546	.050	14	23,137	.058	324	477,683	.050
Class 1	3,053	3,449,099	.064	1,485	1,698,079	.068	4,538	5,147,178	.065
Class 2	213	222,555	.068	112	126,427	.076	325	348,982	.071
Class 3	19	15,279	.076	255	283,352	.074	274	298,631	.074
Class 4	1	1,132	.066	2	1,907	.086	3	3,039	.079
Other	1	1,767	.050	3	3,372	.061	4	5,139	.057
All	3,597	4,144,378	.063	1,871	2,136,274	.069	5,468	6,280,652	.065

[1] Nineteen observations for 4-cylinder Willys are excluded from the tabulation.
[2] Includes reports from employees of Division Office of Bureau of Public Roads and State highway employees.
[3] Includes reports from employees of both the Division and Regional Offices of the Bureau of Public Roads and State highway employees.

ass 4 cars having automatic transmissions e probably the result of an insufficient mber of sample cases.

ypothesis

A hypothesis may be put forth that, if ansmission type and vehicle class are held nstant, a linear relationship exists between el-consumption rates and the percentage of iving at speeds not exceeding 35 miles per ur, and that these lines are parallel within transmission type. However, instead of fitting a straight line for each vehicle class, the data for classes 1, 2, and 3 were averaged. Under the hypothesis, these averages for the combined classes represent points on a line that is parallel to the lines for the separate classes. A straight line was fitted by the method of least squares to the combined class averages. The slope for automatic transmissions was 0.00020; the slope for manual transmissions was 0.00017. Both slopes differed from zero by a significant amount, as determined by the "t" test. The slope indicated that for every increase of 10 percent in mileage of stop-and-go driving, the rate of fuel consumption increased approximately 0.002 of a gallon per mile when transmission and vehicle class were held constant. The lowest rate of fuel consumption per mile of travel should be realized when stop-and-go driving is reduced to zero. However, the absence of stop-and-go driving is often accompanied by an increase in speed that tends to negate the benefit of uninterrupted driving, and some indications (*3*) have been noted that fuel-consumption rates increase when vehicle speeds pass a critical point. This factor may be very important in an analysis of fuel-consumption requirements for travel on some sections of highway.

Year Model

The age of vehicles as indicated by the year of the model also was considered as a factor that affects fuel-consumption rates. Annual data are available on the registered number of passenger cars classified by year model and on the number of vehicles manufactured and sold. Such data can be obtained from manufacturers, trade associations, and official registration records. It had been hoped that the age of the vehicles could provide another factor for use in estimating fuel-consumption, but a sufficiently pronounced relationship was not established in this study. Table 9 and figure 5 contain information that shows the year model of vehicles to have little noticeable effect on fuel-consumption rates when large numbers of vehicles in normal operation are considered.

Foreign Cars

Reports received for foreign cars totaled 522. Of these, 162 reports did not include the number of cylinders; 341 reports represented 4-cylinder cars; 17 reports represented 6-cylinder cars; 2 reports represented 8-cylinder cars. The fuel consumption rate of foreign cars classified as "cylinder unknown" was calculated at 0.037 gallon per mile; the same rate determined for foreign cars having 4 cylinders. Therefore, most of the cars in the cylinders unknown class reasonably may be assumed to have been 4-cylinder vehicles. The average fuel-consumption rate for the 6-cylinder foreign cars was 0.058 gallon per mile; and for the 8-cylinder cars, it was 0.079 gallon per mile. Because only two reports represented foreign cars having automatic transmissions, that factor was not related to fuel-consumption rates for this group of vehicles.

Application of Study Data

One possible application of the fuel-consumption rates determined from this study is illustrated in table 10. All the entries for vehicles and mileage in this table are estimates that had been prepared by the Highway Cost Allocation Study staff of the Bureau of Public Roads.

Table 5.—Data on miles of travel and fuel-consumption rates, gallons per mile, for American cars taken from 1960 reports that did not list number of cylinders

Location and vehicle make class	Automatic transmissions			Manual transmissions		
	Reports	Travel	Fuel	Reports	Travel	Fuel
	Number	*Veh.-mi.*	*Gal./mi.*	*Number*	*Veh.-mi.*	*Gal./mi.*
Arizona: [1]						
Class 0	9	9,550	0.054	25	34,659	0.047
Class 1	200	216,656	.070	242	274,135	.065
Class 2	57	66,681	.072	22	21,355	.071
Class 3	141	157,937	.076	18	20,623	.070
Class 4	10	12,117	.090	0	0	------
Other	1	2,228	.064	0	0	------
All	418	465,169	.073	307	350,772	.064
California: [1]						
Class 0	11	14,578	.047	28	34,319	.048
Class 1	570	683,978	.071	558	677,128	.065
Class 2	173	220,816	.073	77	104,046	.059
Class 3	326	403,316	.077	36	41,660	.075
Class 4	36	46,188	.081	0	0	------
Other	5	5,139	.073	3	4,510	.074
All	1,121	1,374,015	.073	702	861,663	.064
All agencies: [2]						
Class 0	196	268,126	.058	377	546,661	.050
Class 1	4,607	5,239,789	.072	5,338	6,098,441	.065
Class 2	1,404	1,562,462	.076	424	474,383	.068
Class 3	3,013	3,382,399	.079	328	360,914	.074
Class 4	198	238,527	.080	3	3,039	.079
Other	12	13,548	.075	7	9,649	.065
All	9,430	10,704,851	.074	6,477	7,493,087	.065

[1] Includes reports from employees of the Division Office of the Bureau of Public Roads and State highway employees.
[2] The totals shown here for all agencies include the totals from tables 3 and 4.

Table 6.—Fuel-consumption rates in gallons per mile computed as miles per gallon, from 1960 reports for American cars

Vehicle make class	Automatic transmission		Manual transmission	
	Gallons [1] per mile	*Miles per gallon*	*Gallons [1] per mile*	*Miles per gallon*
0	0.058	17.3	0.050	20.1
1	.072	14.0	.065	15.3
2	.076	13.1	.068	14.7
3	.079	12.7	.074	13.5
4	.080	12.5	.079	12.7
Other	.075	13.4	.065	15.4
All	.074	13.5	.065	15.5

[1] Data are the same shown in table 5 for all agencies.

Table 7.—Fuel-consumption rates for American cars, classified by make and transmission class, related to number [1] of reports received and the season of the year: 1960

Season	Fuel-consumption rates						Number of reports					
	Class 0	Class 1	Class 2	Class 3	Class 4	Other	Class 0	Class 1	Class 2	Class 3	Class 4	Other
AUTOMATIC TRANSMISSION												
	Gal./mi.	*Gal./mi.*	*Gal./mi.*	*Gal./mi.*	*Gal./mi.*	*Gal./mi.*						
Spring	0.060	0.073	0.077	0.080	0.078	0.076	42	1,183	383	789	58	3
Summer	.056	.068	.072	.074	.077	.072	59	1,135	339	733	55	5
Fall	.054	.070	.075	.076	.079	------	53	1,076	325	671	38	0
Winter	.063	.076	.083	.084	.087	.077	42	1,213	357	820	47	4
All	.058	.072	.076	.079	.080	.075	196	4,607	1,404	3,013	198	12
MANUAL TRANSMISSION												
Spring	0.050	0.065	0.068	0.073	------	------	95	1,422	114	104	0	0
Summer	.048	.062	.065	.069	.068	.067	95	1,222	100	72	1	3
Fall	.049	.064	.068	.073	------	.079	108	1,192	109	62	0	1
Winter	.054	.069	.073	.080	.088	.060	79	1,502	101	90	2	3
All	.050	.065	.068	.074	.079	.065	377	5,338	424	328	3	7

[1] Nineteen observations for 4-cylinder Willys are excluded from the tabulations.

Table 8.—Number [1] of reports and fuel-consumption rates for American cars related to percentage of mileage driven at speeds of 35 m.p.h. or less: 1960

Mileage driven at speeds of 35 m.p.h. or less	Fuel-consumption rates						Number of reports					
	Class 0	Class 1	Class 2	Class 3	Class 4	Other	Class 0	Class 1	Class 2	Class 3	Class 4	Other
AUTOMATIC TRANSMISSION												
Percent	*Gal./mi.*	*Gal./mi.*	*Gal./mi.*	*Gal./mi.*	*Gal./mi.*	*Gal./mi.*						
0.0–9.9	0.047	0.065	0.069	0.069	0.078	0.066	9	250	76	184	17	2
10.0–19.9	.055	.066	.071	.072	.071	----	28	679	174	401	36	----
20.0–29.9	.050	.068	.073	.075	.075	.052	21	642	194	391	36	1
30.0–39.9	.061	.071	.073	.076	.080	.078	20	551	167	361	26	2
40.0–49.9	.060	.072	.074	.078	.075	.080	25	516	135	328	15	2
50.0–59.9	.061	.073	.076	.080	.088	----	38	553	163	332	16	----
60.0–69.9	.061	.076	.080	.083	.085	----	16	336	99	270	18	----
70.0–79.9	.063	.077	.081	.084	.092	.076	10	298	122	197	10	2
80.0–89.9	.064	.078	.087	.088	.093	.114	11	283	91	193	9	1
90.0–100.0	.058	.080	.088	.090	.092	.074	18	499	183	356	15	2
All	.058	.072	.076	.079	.080	.075	196	4,607	1,404	3,013	198	12
MANUAL TRANSMISSION												
0.0–9.9	0.046	0.061	0.064	0.069	----	----	38	374	18	23	----	----
10.0–19.9	.047	.062	.060	.066	.066	----	70	962	60	34	1	----
20.0–29.9	.049	.063	.064	.070	.067	0.067	56	701	63	36	1	1
30.0–39.9	.048	.064	.065	.074	----	.048	31	647	40	39	----	1
40.0–49.9	.049	.065	.070	.074	----	.050	25	484	55	31	----	1
50.0–59.9	.051	.066	.068	.071	----	----	50	599	41	42	----	1
60.0–69.9	.053	.069	.070	.075	----	----	30	353	37	28	----	----
70.0–79.9	.054	.069	.076	.080	----	.068	26	336	30	16	----	1
80.0–89.9	.054	.070	.077	.087	----	.070	19	297	20	17	----	2
90.0–100.0	.059	.073	.083	.084	.147	.088	32	585	60	62	1	1
All	.050	.065	.068	.074	.079	.065	377	5,338	424	328	3	7

[1] Nineteen observations for 4-cylinder Willys are excluded from the tabulations.

Table 9.—Number [1] of reports and average fuel-consumption rates for American cars, classified by vehicle make, transmission class, and year model: 1960

Year model	Fuel-consumption rates						Number of reports					
	Class 0	Class 1	Class 2	Class 3	Class 4	Other	Class 0	Class 1	Class 2	Class 3	Class 4	Other
AUTOMATIC TRANSMISSION												
	Gal./mi.	*Gal./mi.*	*Gal./mi.*	*Gal./mi.*	*Gal./mi.*	*Gal./mi.*						
1961	0.063	0.068	0.091	0.078	----	----	4	4	2	3	----	----
1960	.053	.067	.073	.077	0.078	----	95	278	104	138	6	----
1959	.062	.070	.077	.077	.090	0.069	43	723	161	309	12	4
1958	.064	.072	.077	.080	.078	.082	11	744	115	256	8	3
1957	.065	.074	.078	.081	.081	.059	22	956	201	419	25	2
1956	.055	.072	.075	.078	.082	.064	6	687	162	446	37	1
1955	.060	.072	.075	.075	.076	.108	9	579	259	569	20	2
1954	.050	.072	.076	.078	.081	----	1	277	101	262	9	----
1953	.065	.073	.077	.080	.075	----	5	210	141	218	29	----
1952	----	.075	.077	.082	.078	----	----	69	50	136	16	----
1951	----	.074	.084	.085	.075	----	----	63	37	96	12	----
Older than 1951	----	.069	.081	.082	.084	----	----	17	71	161	24	----
All	.058	.072	.076	.079	.080	.075	196	4,607	1,404	3,013	198	12
MANUAL TRANSMISSION												
1961	0.048	0.058	0.049	0.057	----	----	1	7	2	3	----	----
1960	.046	.060	.059	.071	----	0.068	196	234	30	10	----	1
1959	.051	.063	.054	.058	----	.061	89	464	42	6	----	3
1958	.056	.063	.062	.074	----	----	32	396	10	7	----	----
1957	.054	.064	.067	.075	----	.068	20	584	11	9	----	1
1956	----	.065	.069	.074	----	----	----	594	22	24	----	----
1955	.055	.065	.073	.070	----	----	8	742	49	36	----	----
1954	.062	.065	.069	.072	----	----	6	509	48	38	----	----
1953	.056	.068	.076	.075	.068	----	9	630	78	65	1	----
1952	.053	.068	.072	.070	----	----	4	306	27	19	----	----
1951	.058	.067	.079	.074	.066	.066	7	240	29	26	1	2
Older than 1951	.078	.068	.072	.080	.147	----	5	632	76	85	1	----
All	.050	.065	.068	.074	.079	.065	377	5,338	424	328	3	7

[1] Nineteen observations for 4-cylinder Willys are excluded from the tabulations.

Table 10.—Estimated fuel-consumption data and fuel-tax yield for passenger cars in United States for calendar 1960

Make class [1]	Number of vehicles	Vehicle-miles (at 9,600 miles per vehicle)	Gallons per mile	Gallons of gasoline	Fuel-tax yield at four cents a gallon (dollars)
AUTOMATIC TRANSMISSION					
Class 0	1,417,710	13,610,016,000	0.058	789,380,900	31,575,236
Classes 1 and 5	19,199,630	184,316,448,000	.072	13,270,784,300	530,831,372
Class 2	7,323,744	70,307,942,400	.076	5,343,403,600	213,736,144
Class 3	11,634,682	111,692,947,200	.079	8,823,742,800	352,949,712
Class 4	1,474,140	14,151,744,000	.080	1,132,139,500	45,285,580
Foreign	---	---	---	---	---
Total	41,049,906	394,079,097,600	.075	29,359,451,100	1,174,378,044
MANUAL TRANSMISSION					
Class 0	2,126,565	20,415,024,000	0.050	1,020,751,200	40,830,048
Classes 1 and 5	12,799,754	122,877,638,400	.065	7,987,046,500	319,481,860
Class 2	1,830,936	17,576,985,600	.068	1,195,235,000	47,809,400
Class 3	1,292,742	12,410,323,200	.074	918,363,900	36,734,556
Class 4	163,793	1,572,412,800	.079	124,220,600	4,968,824
Foreign	2,166,898	20,802,220,800	.037	769,682,200	30,787,288
Total	20,380,688	195,654,604,800	.061	12,015,299,400	480,611,976
AUTOMATIC AND MANUAL TRANSMISSIONS					
Class 0	3,544,275	34,025,040,000	0.053	1,810,132,100	72,405,284
Classes 1 and 5	31,999,384	307,194,086,400	.069	21,257,830,800	850,313,232
Class 2	9,154,680	87,884,928,000	.074	6,538,638,600	261,545,544
Class 3	12,927,424	124,103,270,400	078	9,742,106,700	389,684,268
Class 4	1,637,933	15,724,156,800	.080	1,256,360,100	50,254,404
Foreign	2,166,898	20,802,220,800	.037	769,682,200	30,787,288
Total	61,430,594	589,733,702,400	.070	41,374,750,500	1,654,990,020

[1] Other class American cars have been included in class 5 figures.

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VOL. 32, NO. 6

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Public Roads

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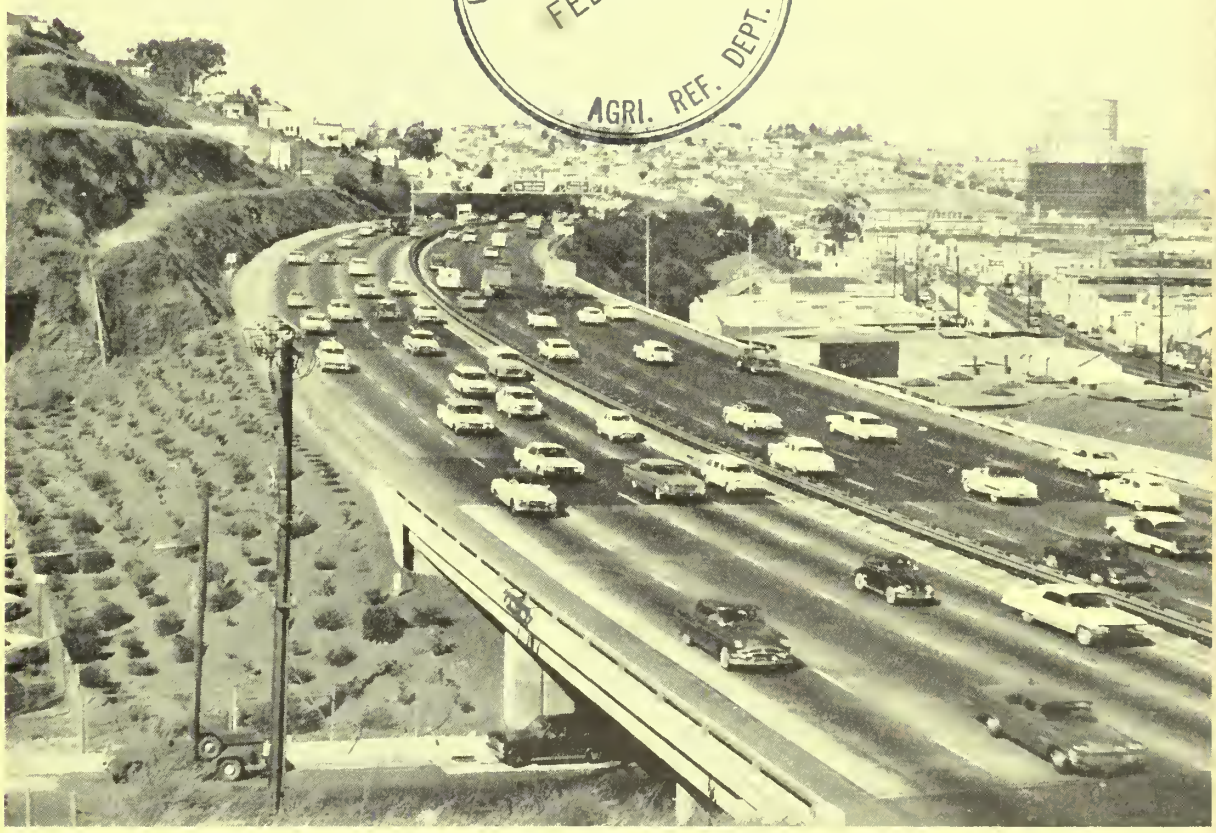

Bayshore Freeway (Bypass 101) showing 8 traffic lanes between Daly City and San Francisco, California, looking northeast from Industrial Street, Interstate Route 5.

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Contents of this publication may be reprinted. Mention of source is requested.

Structural Behavior of Flexible Pavement

Analysis of Rigid-Plate Bearing Tests on Full-Size Test Sections

Y THE PHYSICAL RESEARCH DIVISION
UREAU OF PUBLIC ROADS

By[1] ALVIN C. BENKELMAN[2] and STUART WILLIAMS,
Chief, Flexible Pavement Section

This article presents the first comprehensive analysis of the data collected during an investigation of the structural behavior of flexible pavement conducted during the 1945–1950 period at Hybla Valley, near Alexandria, Virginia. Other articles have been published on the objectives of the investigation and the procedures, techniques, and equipment employed. Also, the refined data have been published and portions of these data have been analyzed by others.

The investigation at Hybla Valley, initiated by the Bureau of Public Roads, is believed to have been the most comprehensive experiment of this type ever conducted. Although the original objectives of the study were not attained in their entirety, the project has stimulated further research related to flexible pavement behavior and design. The new testing procedures and instrumentation developed were subsequently used by State highway departments and staffs of the WASHO and AASHO Road Test projects.

The comprehensive analysis presented in this article is expected to stimulate further interest in the application of the Hybla Valley test data to the design of flexible pavements.

Introduction

N ANALYSIS of data obtained in a
A five-year investigation of the structural
havior of flexible pavements is presented
this article. During the 1945–1950 period,
rid-plate bearing tests were conducted on
ll-size test sections at Hybla Valley near
exandria, Va., by the Bureau of Public
ads with the cooperation of the Asphalt
stitute and the Highway Research Board.
velopment of data on fundamental re-
ions between load and the thickness of
xible pavement structures was one of the
mary objectives of this investigation.

Information relative to this study of flexible pavements has been published in articles appearing in previous issues of PUBLIC ROADS magazine (*1*, *2*)[3] and publications issued by the Highway Research Board (*3*). Objectives of the investigation and the design and construction of the test sections, and the development of testing apparatus, techniques, and procedures were set forth in references *1* and *2*. Collected data on test findings have been presented in HRB Special Report 46 (*3*), to encourage independent analyses. These data included comprehensive tabulations of most of the refined plate-bearing test data, detailed descriptions of the test procedures, and descriptive summaries of the data refinement processes. Portions of these data have been analyzed in other publications (*4*, *5*).

Many different procedures can be used for plate-bearing tests and the data obtained may be analyzed in several ways (*3*). Most of the data obtained from use of three different test procedures have been analyzed for this article. These analyses—based on selected vertical movement criteria—show principally the load-bearing capacity of different combinations of pavement structure components on a given subgrade soil and of the subgrade itself. The data included in HRB Special Report 46 that have been analyzed for this article have not been repeated here; however, references have been made to tabulations of pertinent data at the time of their specific analysis.

For convenience, this article includes summaries of previously published information regarding the test facilities, the laboratory and field testing of materials and soil, and the test procedures used in the Hybla Valley investigation.

Findings

Flexible pavements to be adequate for carrying prevailing traffic must be designed to act in an essentially elastic manner. To meet this requirement each component of the pavement must possess (1) sufficient inherent stability to resist distortion within itself and (2) the density necessary to resist consolidation. The findings from the analysis of test data from structural behavior of pavement should be helpful to those concerned with designing and building flexible pavements.

Data from a series of Repetitional Tests showed that the thickness of pavement structure required to support a unit load of 80 p.s.i. varies approximately as the total load to the 0.4 power. At this required thickness (3 inches of AC surface plus granular base in this series of tests) no detrimental or nonelastic movement of the subgrade soil occurred.

Some of the other more important findings are given in the following paragraphs.

• When the subgrade soil was tested by having the load applied directly through rigid plates, the same degree of elastic deflection was not developed in the subgrade soil as when the load was distributed to the subgrade through the overlying pavement structure.

Presented at the International Conference on Structural
ign of Asphalt Pavement, University of Michigan, Ann
or, Mich., August 1962.

Mr. Benkelman is now retired. During the study
ented in this article he was employed by the Bureau of
lic Roads as a Highway Research Engineer and later
ed as the Flexible Pavement Research Engineer for the
SHO Road Test.

[3] References indicated by italic numbers in parentheses have been listed on page 140.

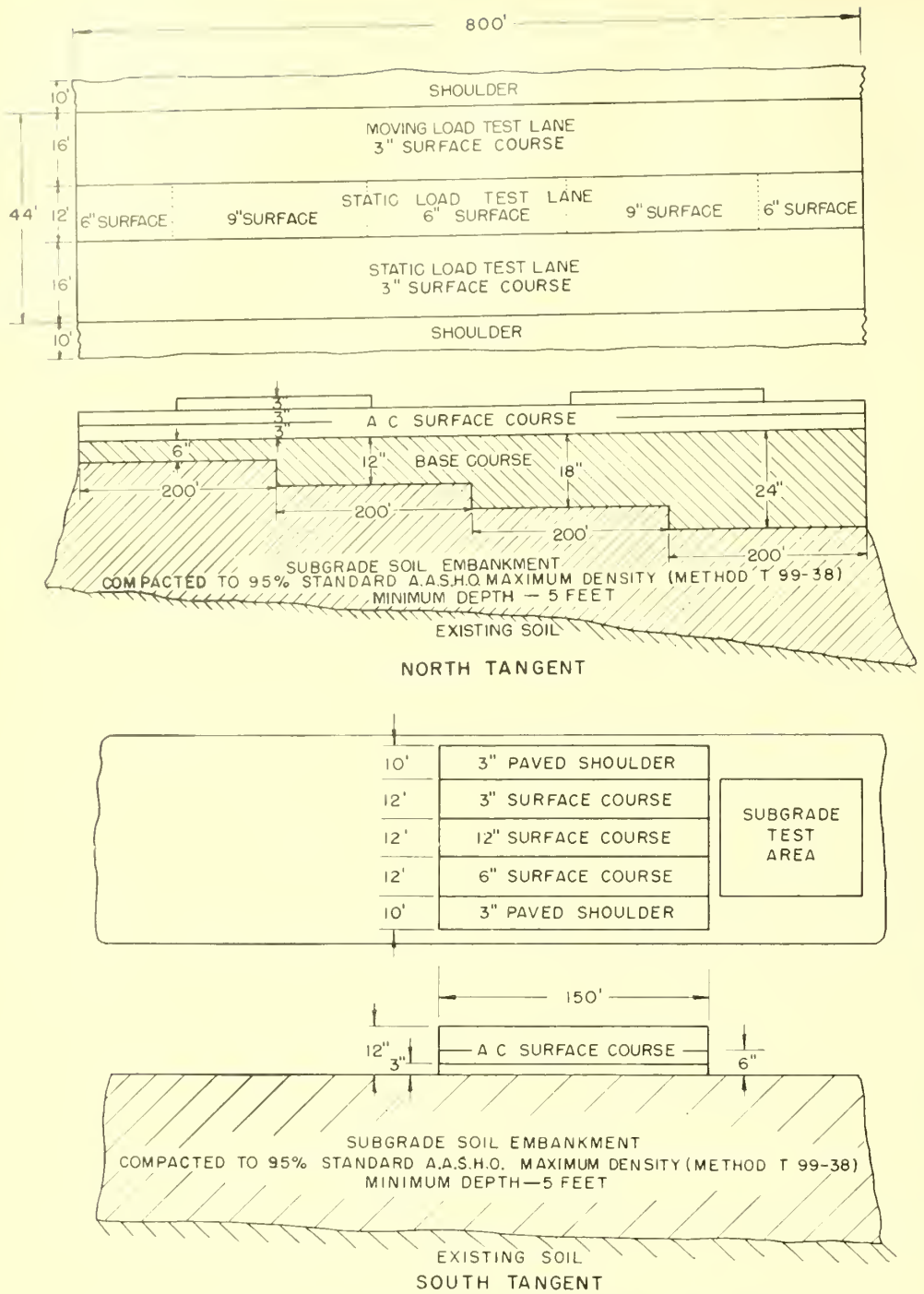


Figure 1.—Plan and profile of north and south tangents of test track.

Table 1.—Thickness of test sections

Test section	Thickness— Surface course	Base course	Total structure
NORTH TANGENT			
Number	*Inches*	*Inches*	*Inches*
1	3	6	9
2	3	12	15
3	3	18	21
4	3	24	27
5	6	6	12
6	6	12	18
7	6	18	24
8	6	24	30
9	9	6	15
10	9	12	21
11	9	18	27
12	9	24	33
SOUTH TANGENT			
13	3	0	3
14	6	0	6
15	12	0	12
16	0	0	0

Test Facilities

The tests on structural behavior of flexil pavements were conducted on two 800-fo tangents of an oval test track. The plan a profile of the north and south tangents of t test track are shown in figure 1. A tabulati of the thicknesses of the test sections is giv in table 1. The test sections located on t north tangent consisted of either 3-, 6-, or inch thicknesses of bituminous concrete su faces each laid respectively on 6-, 12-, 18-, a 24-inch granular base courses that had be constructed on a uniform clay-soil embar ment. The test sections on the south tange consisted of either 3-, 6-, or 12-inch bitumino concrete courses laid directly on the unifo clay-soil embankment, which was of the sa type as the clay soil in the embankment the north tangent. Tests were also ma directly on the subgrade soil of the so tangent in an area reserved for these spec tests. Test sections were each a minimum 100 feet in length and had a minimum wi of 12 feet.

The subgrade was constructed of a unifo A–7–6 soil in 4-inch layers compacted t minimum of 95 percent of the maxim density obtained by *Standard Laborat Method of Test for The Compaction o Density of Soil, AASHO Designation: T 99–* in *Highway Materials*, Part II, 1947, 1 212–213. The minimum height of embar ment was 5 feet. The base course was uniform dense-graded mixture of soil, sa and gravel conforming closely to grading E of *Standard Specifications for Materials Stabilized Base Course, AASHO Designati M 56–42*, in *Highway Materials*, Part I, 19 pp. 35–36. It was constructed in 3-in compacted layers to a minimum, in-place d density of 136 pounds per cubic foot (p.c.

The bituminous concrete surface course w a dense-graded, hot, plant-mix type confor ing to specifications of the Asphalt Institu (*6*). Mix No. IV was used for all 3-in

- Test data showed a constant and orderly interrelation of the effect of unit load, diameter of test plate, and thickness of pavement structure.
- When average surface course temperatures exceeded 75° F., the granular base course appeared to be more effective in supporting load applied through a rigid plate than an equal thickness of AC only. But, when the temperatures of the surface course fell below 75° F., the AC surface was more effective than the granular base in supporting the load applied through a rigid plate.
- The ability of the pavement structure to support load was appreciably greater when the average pavement temperature was between 40° F. and 50° F. than when the pavement temperature was between 75° F. and 95° F.
- The surcharge provided by the AC surface course appeared to have had little effect upon the ability of the base course to support load applied through rigid plates. Likewise, the surcharge provided by the surface plus base course had little effect on the load-supporting capacity of the subgrade.
- The unit load supported by the subgrade soil at a given deflection decreased at a diminishing rate as the size of the loaded area was increased up to that of a bearing plate 84 inches in diameter—the maximum size of plate used in these tests.

Table 2.—Summary of principal features of test procedures

	Accelerated Test—		Repetitional Test—		Incremental-Repetitional Test
	Part *a*	Part *b*	Part *a*	Part *b*	
Number of different loads.	3		4	1	4
Successive deflections, approximately, inches	0.20 0.30 0.40	(1)	(1)		0.125 0.250 0.375 0.500
Load, p.s.i.	(1)	(1)	16 32 48 64	80	(1)
Number of load applications or release.	1	Continuous to end of test.	1	75	5
Duration of each load application or release.	Until movement decreased to 0.001 inch per 15 sec.		Until movement decreased to 0.001 inch per 15 sec.	1 minute	Until rate of movement decreased to 0.001 inch per minute.
Plate diameter, inches	12 18 24 30	12 18 24 30	6 to 42 in 3-inch increments.	6 to 42 in 3-inch increments.	12 18 24 30

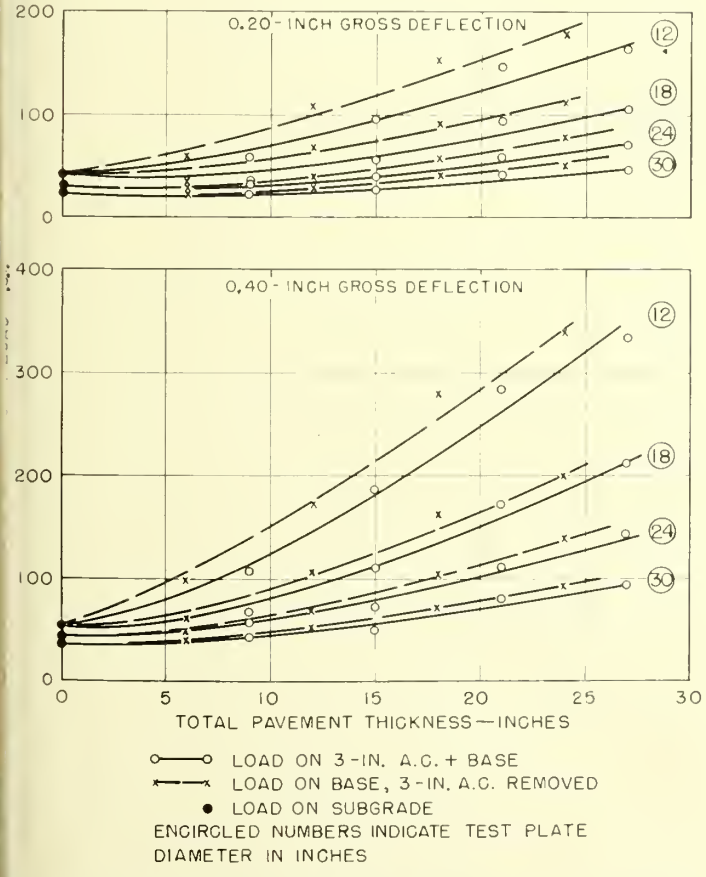
[1] Depended upon test conditions.

urface courses and for the top 3 inches of the , 9-, and 12-inch surface courses. Mix No. was used for the lower portions of the 6-, , and 12-inch surface courses. The test ctions were built with great care, extreme ecautions being taken to ensure uniformity composition and compaction of the subgrade, and in composition, compaction, and thickness of the pavement structure components. Detailed descriptions of the test facilities—including the layout of test sections—and materials and construction methods used are contained in previous publications (*1*, *2*, and *3*).

Laboratory and Field Testing of Materials

In connection with the loading tests, representative samples of the pavement structure components and the subgrade were obtained periodically from most of the test sections for laboratory analysis. In addition, in-place density and moisture determinations were made of the base course, and undisturbed cores of the subgrade were obtained for density and moisture content determinations.

Results of preliminary tests emphasized the importance of the effect of temperature of the bituminous (AC) surface course at the time of testing on the ability of the different sections to support load. Consequently, thermocouples were installed at several different depths in the pavement structure of representative test sections, and temperature measurements were recorded routinely. Precise control of the temperature of the bituminous surface courses during a series of loading tests was of course impossible. However, by limiting the testing season from mid-May to mid-September of each year and by shading the pavement in the vicinity of the test site, the average temperature of the bituminous surface could be controlled within what seemed to be reasonable limits of about 75° F. and 95° F. This procedure was followed for all tests in which it was desired


0.20-INCH GROSS DEFLECTION
0.40-INCH GROSS DEFLECTION
TOTAL PAVEMENT THICKNESS—INCHES
LOAD ON 3-IN. A.C. + BASE
LOAD ON BASE, 3-IN. A.C. REMOVED
LOAD ON SUBGRADE
ENCIRCLED NUMBERS INDICATE TEST PLATE DIAMETER IN INCHES

gure 2.—Load-thickness curves of the pavement and base course based on gross deflection of the medium tested—3-inch AC surface sections.

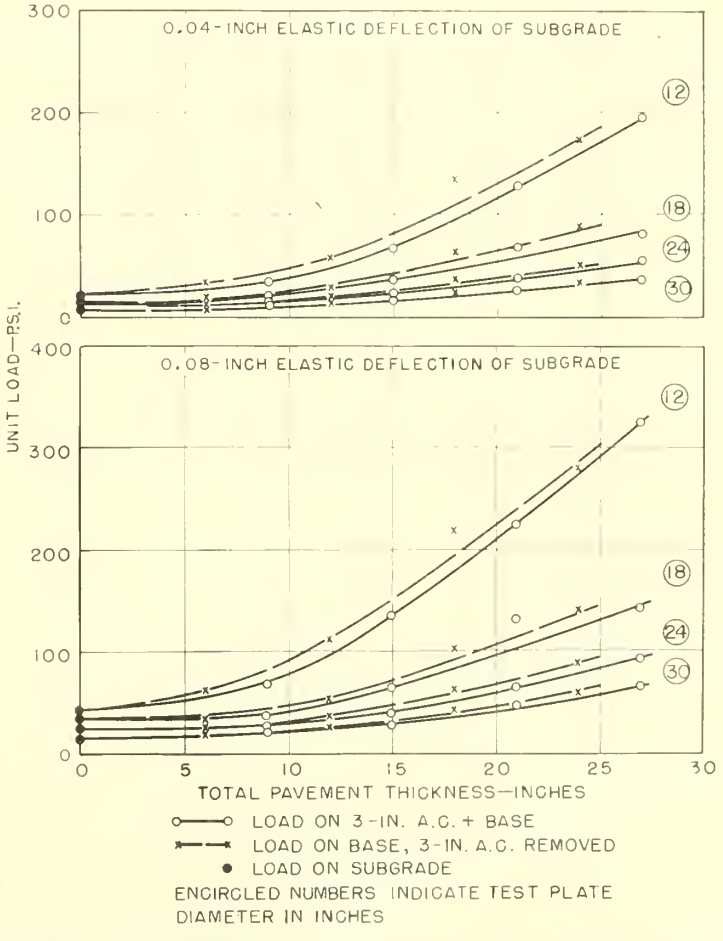

0.04-INCH ELASTIC DEFLECTION OF SUBGRADE
0.08-INCH ELASTIC DEFLECTION OF SUBGRADE
UNIT LOAD—P.S.I.
TOTAL PAVEMENT THICKNESS—INCHES
LOAD ON 3-IN. A.C. + BASE
LOAD ON BASE, 3-IN. A.C. REMOVED
LOAD ON SUBGRADE
ENCIRCLED NUMBERS INDICATE TEST PLATE DIAMETER IN INCHES

Figure 3.—Load-thickness curves of the pavement and base course based on elastic deflection of the subgrade—3-inch AC surface sections.

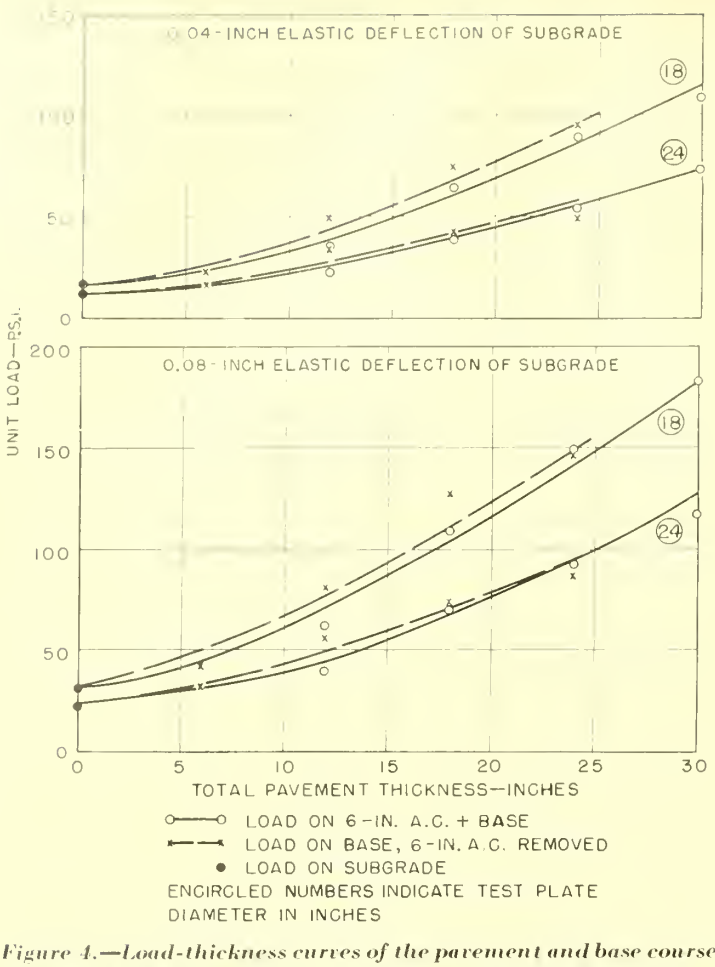

0.04-INCH ELASTIC DEFLECTION OF SUBGRADE
0.08-INCH ELASTIC DEFLECTION OF SUBGRADE
UNIT LOAD—P.S.I.
TOTAL PAVEMENT THICKNESS—INCHES
LOAD ON 6-IN. A.C. + BASE
LOAD ON BASE, 6-IN. A.C. REMOVED
LOAD ON SUBGRADE
ENCIRCLED NUMBERS INDICATE TEST PLATE DIAMETER IN INCHES

Figure 4.—Load-thickness curves of the pavement and base course based on elastic deflection of the subgrade—6-inch AC surface sections.

to control the temperature within these limits. For tests made on the granular base and on the subgrade, the temperature of the materials within the range obtaining during any of the tests was not considered a significant factor.

Plate-Bearing Test Procedures

Definitions of pertinent terms and a detailed description of the test procedures used have been reported previously (*1*, *2*, and *3*). However, to facilitate comprehension of this article, a summarization has been included. The following listed definitions apply.

Pavement.—The entire structure including the surface course, and the base course where present. Movements of the pavement included those occurring in the subgrade.

Surface course.—The asphaltic concrete (AC) course that rested on the base course or on the subgrade.

Base course.—The granular aggregate course that rested on the subgrade.

Subgrade.—The soil embankment underlying the base or surface course.

Gross deflection.—The total vertical movement, both elastic and inelastic, of the pavement, of the base plus subgrade, or of the subgrade only caused by the application of a single load or more than one load.

Settlement.—The permanent or inelastic vertical displacement caused by the application of a single load or the cumulative permanent displacement caused by the application of more than one load.

Elastic deflection.—The portion of the gross deflection of the pavement, the base plus subgrade, or the subgrade only that was recovered upon release of the load.

Early in the investigation it was realized that the data could be greatly augmented and the analysis enhanced or broadened by obtaining measurements of the vertical movements, not only on the surface being loaded but also simultaneously at other levels in the pavement structure. After considerable experimental work, a reliable and accurate method was developed (*3*). Consequently, for all the tests reported here, when tests were made on the surface course simultaneous measurements of movement were made at the surface, at the top of the base course, and at the top of the subgrade. When tests were made on the base course simultaneous measurements were made on the top of the base course and the top of the subgrade.

At the time of this investigation no standard or generally accepted procedure for conducting rigid-plate bearing tests had been developed. Consequently, one of the objectives of the rather elaborate preliminary field testing program was the development of such a procedure. Most of the data obtained and reported herein were obtained by either one or the other of two newly developed procedures, the Accelerated Test and the Repetition Test. However, some additional data obtained by a third procedure, the Incremental Repetitional Test, also have been included. The principal features of each of these tests are shown in table 2. The standards developed for conducting these tests were meticulously adhered to.

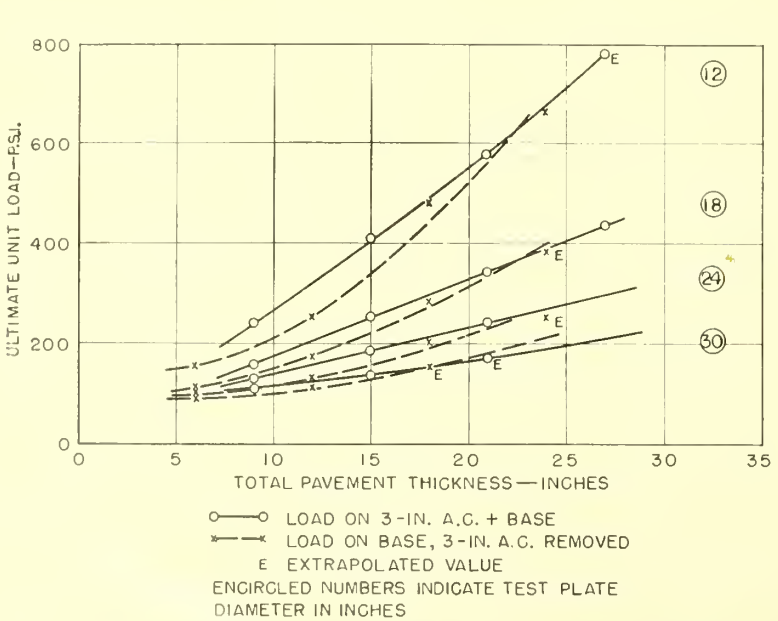

ULTIMATE UNIT LOAD—P.S.I.
TOTAL PAVEMENT THICKNESS—INCHES
LOAD ON 3-IN. A.C. + BASE
LOAD ON BASE, 3-IN. A.C. REMOVED
E EXTRAPOLATED VALUE
ENCIRCLED NUMBERS INDICATE TEST PLATE DIAMETER IN INCHES

Figure 5.—Ultimate load-thickness curves of the pavement and base course—3-inch AC surface sections.

The Accelerated Test

The Accelerated Test consisted of two parts, the incremental (part *a*) and the accelerated (part *b*). The incremental part provided for the application and release, once each, of three individual loads of increasing magnitude. The period of application or release was maintained until the rate of movement slowed

o 0.001 of an inch in 15 seconds. The magnitudes of the loads were selected by estimation so as to produce gross deflections of approximately 0.20, 0.30, and 0.40 of an inch for each of the three loads, respectively. Since the loads were necessarily estimated, it was intended that the gross deflections mentioned should be considered merely goals that were sought rather than specific gross deflections that were to be attained.

After release of the third load and after the movement-time criterion of the incremental part of the test had been satisfied, the accelerated part of the test was begun. It consisted of the continuous application of a load of increasing magnitude, the rate of load application being controlled so as to produce a rate of vertical movement of the surface under test of 0.5 of an inch per minute. The application of the load was continued until: (1) the material was unable to support a further increase in the load, (2) the gross deflection exceeded 2.0 inches, or (3) the total reaction load had been utilized. In nearly all of the tests, condition (1) was reached first; the maximum load applied to reach this condition was called the ultimate load. The rate of load application in the incremental part of the Accelerated Test was comparatively high, the total elapsed time required to cause the selected maximum gross deflection of 2.0 inches being only 4 minutes. Also, the maximum gross deflection in the incremental part of this test was much greater than in any of the other types of tests.

The Repetitional Test

The Repetitional Test was developed in an effort to determine whether the elastic action of a flexible pavement structure might serve as an acceptable criterion of its load-supporting capacity. Like the Accelerated Test, it consisted of two parts, part *a* being similar to the first part (incremental) of the Accelerated Test. In part *a* of the Repetitional Test, 16-, 32-, 48-, and 64-p.s.i. loads were applied and released once each, and the resultant vertical movements were measured. The movement-time criterion was the same as that of the Accelerated Test. Part *b* required the application and release 75 times of a constant unit load of 80 p.s.i. For each repetition, the load was applied and removed alternately for 1-minute periods of time.

The principal objective of the Repetitional Test was to determine the magnitude of total load for a given unit load or contact pressure under which the subgrade would act in an essentially elastic manner when the load was applied repetitiously to the surface. Since it was desired that the magnitude of the unit test load approximate the tire contact pressures of the heavier commercial vehicles, a unit load of 80 p.s.i. was used as standard. Thus the magnitude of the total applied load varied directly as the area of the bearing plate. Plates ranging from 6 to 42 inches in diameter in 3-inch increments were used on the different test sections. The range in total applied load was, therefore, from 2.26 to 110.8 thousand pounds deadweight load (kip).

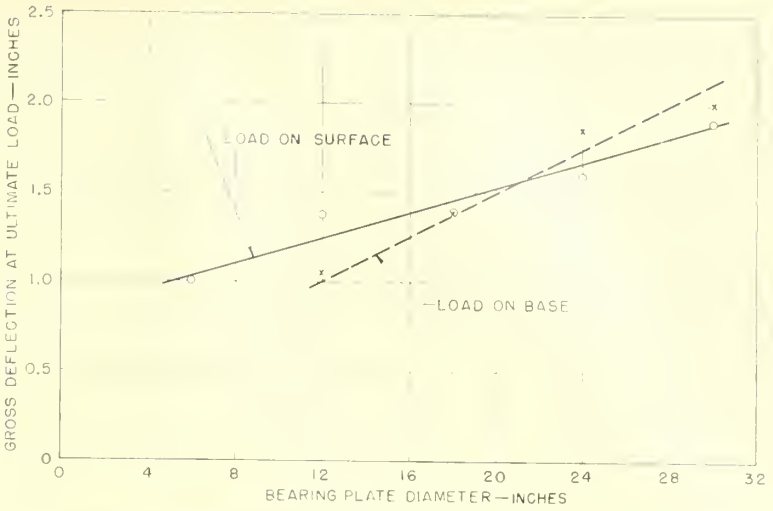


Figure 6.—Deflection-plate diameter curves at ultimate load.

The Incremental-Repetitional Test

The Incremental-Repetitional Test provided for the application of four loads of increasing magnitude, each of which was applied and released five times. Each load was maintained until the rate of movement of the surface under test slowed to 0.001 of an inch per minute. After release of the load, it was not reapplied until the rate of recovery had diminished to the same rate of movement. The loads were selected by estimation so as to cause gross deflections of about 0.125, 0.250, 0.375, and 0.500 of an inch prior to release of the fifth application of each load; as for the incremental part of the Accelerated Test, the gross deflections specified were intended to be considered as guides only.

Analysis of Accelerated Test Data

The data for the Accelerated Tests on the surface and base courses of test sections 1 to 12 inclusive are shown in table 4 and corresponding data for tests on the subgrade are shown in table 5 of reference (*3*). Figures 2 to 4 inclusive show relations between unit load and total pavement structure thickness for the various movement criteria for each of four bearing plate sizes. The data in these figures

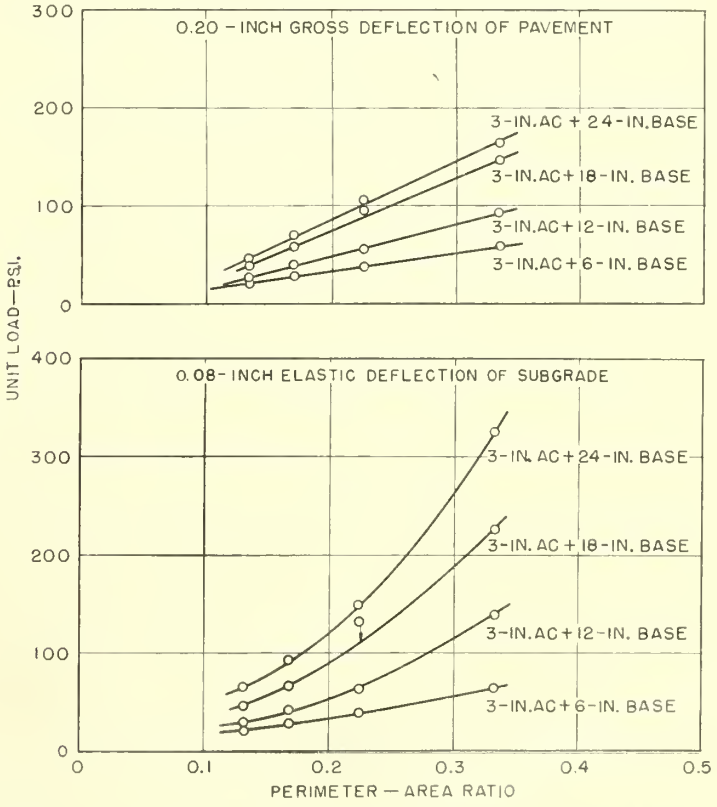


Figure 7.—Load-perimeter area ratio curves based on gross deflection of the pavement and on elastic deflection of the subgrade.

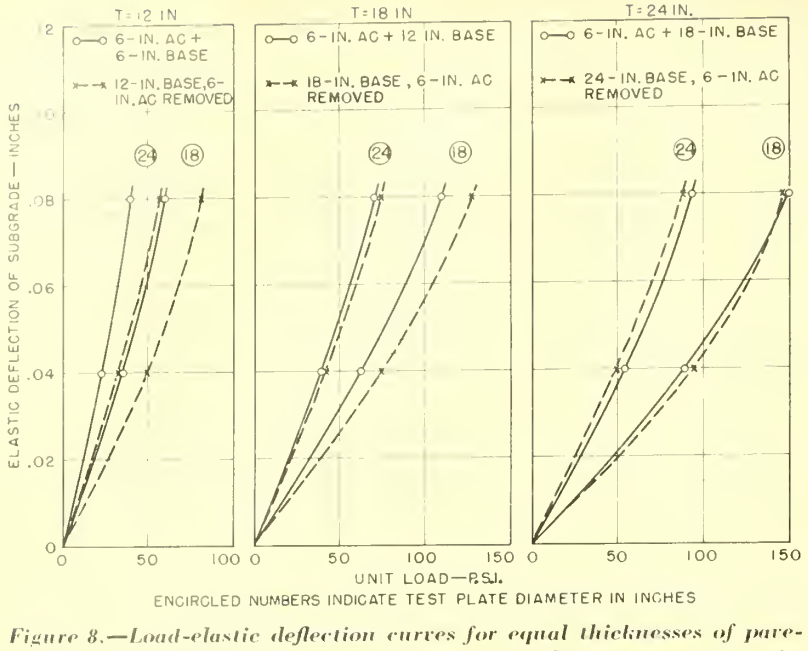

T=12 IN
6-IN. AC + 6-IN. BASE
12-IN. BASE, 6-IN. AC REMOVED
T=18 IN
6-IN. AC + 12 IN. BASE
18-IN. BASE, 6-IN. AC REMOVED
T=24 IN.
6-IN. AC + 18-IN. BASE
24-IN. BASE, 6-IN. AC REMOVED
ELASTIC DEFLECTION OF SUBGRADE—INCHES
UNIT LOAD—P.S.I.
ENCIRCLED NUMBERS INDICATE TEST PLATE DIAMETER IN INCHES

Figure 8.—Load-elastic deflection curves for equal thicknesses of pavement composed of either 6-inch AC surface plus base course or the base course alone.

were obtained from the incremental part of the test. The load data plotted in figure 2 are for gross deflections of 0.20 and 0.40 of an inch of the tested medium (pavement, base, or subgrade) and those in figures 3 and 4 are for elastic deflections of the subgrade of 0.04 and 0.08 of an inch. The gross deflections are given in column 4 in table 4, and in column 3 in table 5 of reference (*3*). The elastic deflections are tabulated in columns 9 and 7 in tables 4 and 5 respectively of the same publication.

In figures 2 through 4, load curves are shown for the granular base course alone acting as a structure, and for pavement structure composed of a bituminous concrete surface course plus the base. For test results shown in figures 2 and 3, the thickness of the surface course was 3 inches and for those in figure 4 it was 6 inches. The notation in the figures, "Load on base, 3-inch AC removed," refers to data for tests made on the base course after removal of the overlying surface course.

In each of the figures, the data plotted on the ordinate (zero thickness of pavement structure) represent those for tests made directly on top of the subgrade. The curves fitted to the plotted data for each of the test plates might be considered to originate at these subgrade points. These curves for the different plates and movement criteria show the extent of the ability of the pavement

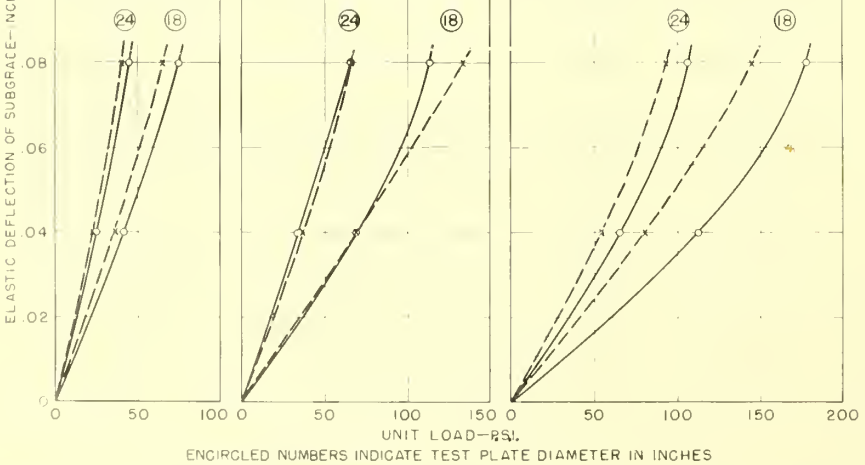

T=15 IN.
9-IN. AC + 6-IN. BASE
3-IN. AC + 12-IN BASE
T=21 IN.
9-IN. AC + 12-IN. BASE
3-IN. AC + 18-IN. BASE
T=27 IN.
9-IN. AC + 18-IN. BASE
3-IN AC + 24-IN BASE
ELASTIC DEFLECTION OF SUBGRADE—INCHES
UNIT LOAD—P.S.I.
ENCIRCLED NUMBERS INDICATE TEST PLATE DIAMETER IN INCHES

Figure 9.—Load-elastic deflection curves for equal thicknesses of pavement composed of either 9-inch AC surface plus base course or 3-inch AC surface plus base course.

structure to support load as the thickness of the structure is increased. In general, the curves are parabolic in character and fit the data reasonably well.

With reference to test results depicted in figures 2 and 3, the following comments are made.

- The effect of thickness of pavement was orderly and consistent. The plotted points define lines that are slightly curved or parabolic and indicate that the unit load supported by a given plate varied as an exponential function of the thickness. Little difference existed in the shape of the curves for the 3-inch AC surface plus base and their shape for an equal thickness of base alone.
- Although the slopes of the curves for the smaller plates are steeper than those for the larger plates, little difference, generally, occurred with the increase in unit load on a percentage basis for any of the plates when computations were made for the same range in thickness.
- The data indicated that, for any given total thickness, the base course as a structure was somewhat more effective in supporting load than an equal thickness of combined surface and base course. This was true for all criteria employed to express load support. However, the difference in indicated effectiveness of the two combinations of structure to support load decreased appreciably as the size of the test plate was increased. In fact, for elastic deflections of the subgrade of 0.04 and 0.08 of an inch, as shown in figure 3, the relations of load to thickness for the plate 30 inches in diameter were practically the same, regardless of the makeup of the structure.

Curves similar to those shown in figures 2 and 3 can also be plotted for a given elastic deflection of the pavement structure. The comments in the three preceding paragraphs also applied when the elastic deflection of the pavement structure was used as a criterion.

The curves presented in figure 4 are similar to those shown in figure 3, except that they apply to the 6-inch AC surface plus base and to the base course after removal of the AC surface. The curves shown are only for the 18- and 24-inch plates. The indications of these data are much the same as those for the 3-inch AC surface plus base and the base alone that is, the base course alone was more effective, generally, in supporting load than an equal thickness of surface and base course combined, but the difference was appreciably less pronounced. Similar curves showing the same trends can be developed for the pavement consisting of a 9-inch AC surface plus base.

The data for figure 5 were obtained by part *b* of the Accelerated Test procedure and are tabulated in column 7 of table 4, reference (*3*). The load was applied continuously at a comparatively rapid rate and the gross deflection at the end of the test was much greater than that obtained in the other test procedures. In figure 5, the ultimate unit load data for each of the sections consisting of 3 inches of AC surface plus base and for the base course under this surface have been plotted against the total thickness of the structure. The

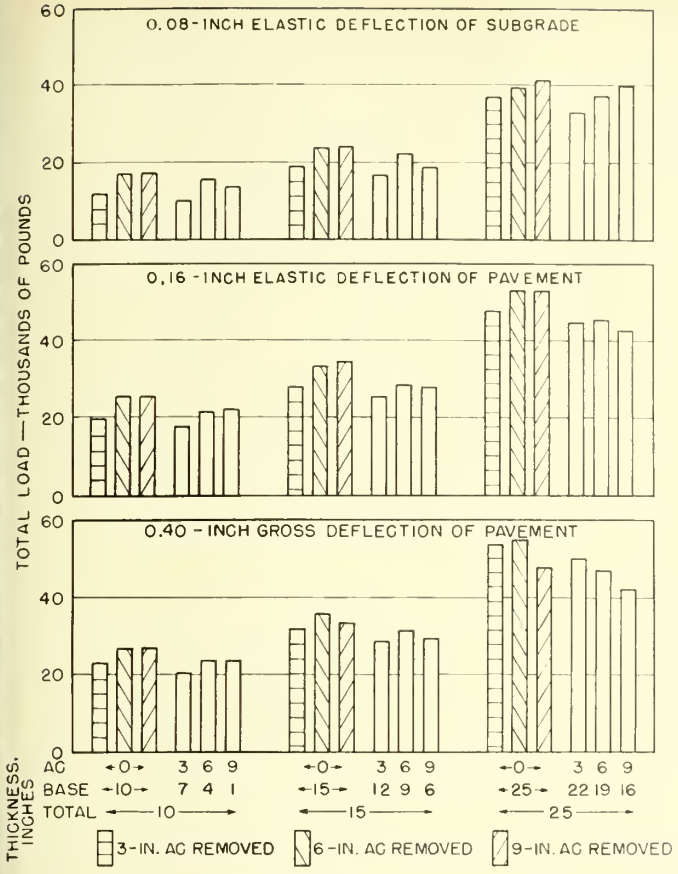


Figure 10.—Summary of Accelerated Test data obtained in tests made with the bearing plate 18 inches in diameter.

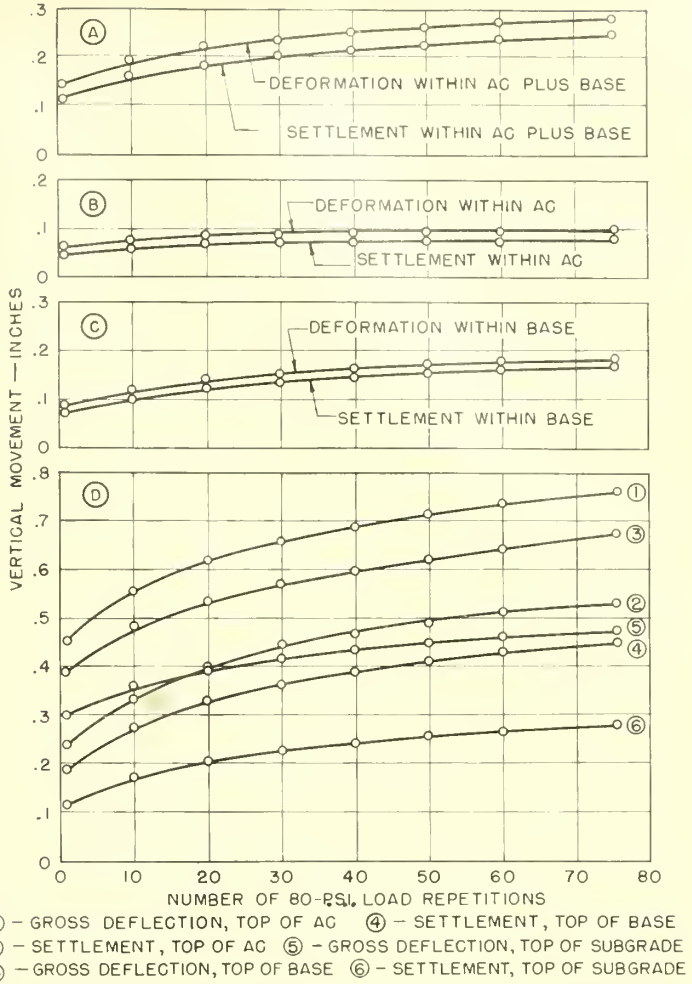

DEFORMATION WITHIN AC PLUS BASE
SETTLEMENT WITHIN AC PLUS BASE
DEFORMATION WITHIN AC
SETTLEMENT WITHIN AC
DEFORMATION WITHIN BASE
SETTLEMENT WITHIN BASE
VERTICAL MOVEMENT — INCHES
NUMBER OF 80-PSI LOAD REPETITIONS
① — GROSS DEFLECTION, TOP OF AC
② — SETTLEMENT, TOP OF AC
③ — GROSS DEFLECTION, TOP OF BASE
④ — SETTLEMENT, TOP OF BASE
⑤ — GROSS DEFLECTION, TOP OF SUBGRADE
⑥ — SETTLEMENT, TOP OF SUBGRADE

Figure 11.—Typical Repetitional Test data for tests in which progressive vertical movement of all pavement components and of the subgrade occurred.

extrapolated data were computed for either the condition in which the available reaction load or the range of the micrometer dials (2.0 inches) was exceeded. The general characteristics of the relations shown in figure 5 are much the same as the curves in figures 2, 3, and 4. However, for total pavement thicknesses of up to about 22 inches, the combination of surface plus base was more effective than the granular base course alone in resisting failure or in postponing the breakdown in resistance.

Column 7 of table 4 in reference (*3*) lists the gross deflections of the pavement structures (surface, base, and subgrade) at the ultimate (maximum) loads. These deflections were recorded when the surface failed in shear around the perimeter of the bearing plate or when the resistance of the material was overcome. These deflections have been plotted in figure 6 as a function of the diameter of the bearing plate. They represent the averages respectively for all the tests made with the 12-, 18-, 24-, and 30-inch diameter plates on all of the pavement sections. Some special tests were made on the surface of the pavement in which a plate 6 inches in diameter was used, and the average gross deflections obtained from these tests were also plotted. The linear curves fitted to the plotted points are reasonably well defined. They indicate that for the smaller plates the gross deflection at ultimate load for the AC surface plus base was greater than that for the base course alone; whereas, for the larger plates, the reverse was true. The range in gross deflections at ultimate load for loading either on the surface or base was from about 1 to 2 inches for the range in plate sizes used. Usually, the AC surface ruptured or failed in shear only when the 6- and 12-inch diameter plates were used in the test. Visual observations and a few radial deflection measurements showed that the curvature of the surface was much flatter when the test was made on the larger plates. This difference in curvature was probably responsible for the marked failure of the surface in the tests in which the smaller bearing plates were used.

Some of the data included in figures 2 and 3 was replotted in figure 7 in order to show more clearly the effect that the size of the bearing plate had on structural strength of the pavement. In figure 7, the unit load was plotted as a function of the perimeter-area ratio of the bearing plate, a ratio that increased with a decrease in diameter of the bearing plate. The upper part of the figure shows the curves for a gross pavement deflection of 0.20 of an inch, and the curves shown in the lower part of the figure are for an 0.08-inch elastic deflection of the subgrade. A consistent and well-defined effect of size of the bearing plate is shown by these test data. They indicated that the ability of the structure to support a given unit load diminished as the thickness of the pavement was decreased and the size of the bearing plate was increased. This effect has been demonstrated in other studies and is consistent with theories that are in use. The curves based on gross deflection were linear and those based on elastic deflection tended to be curvilinear in shape. The deviation from linearity became more pronounced as the overall thickness of pavement structure was increased and as the diameter of the bearing plate was decreased.

In figure 8, test data show the relations between unit load and elastic deflection of the subgrade for tests made with two sizes of plates on pavement structures that had overall thicknesses respectively of 12, 18, and 24 inches. For one series of tests the pavement structure consisted of a 6-inch AC surface plus base and for another the structure was the base alone after the 6-inch surface course directly beneath the test plate had been removed. In figure 9, similar relations are shown for tests made with two sizes of plates

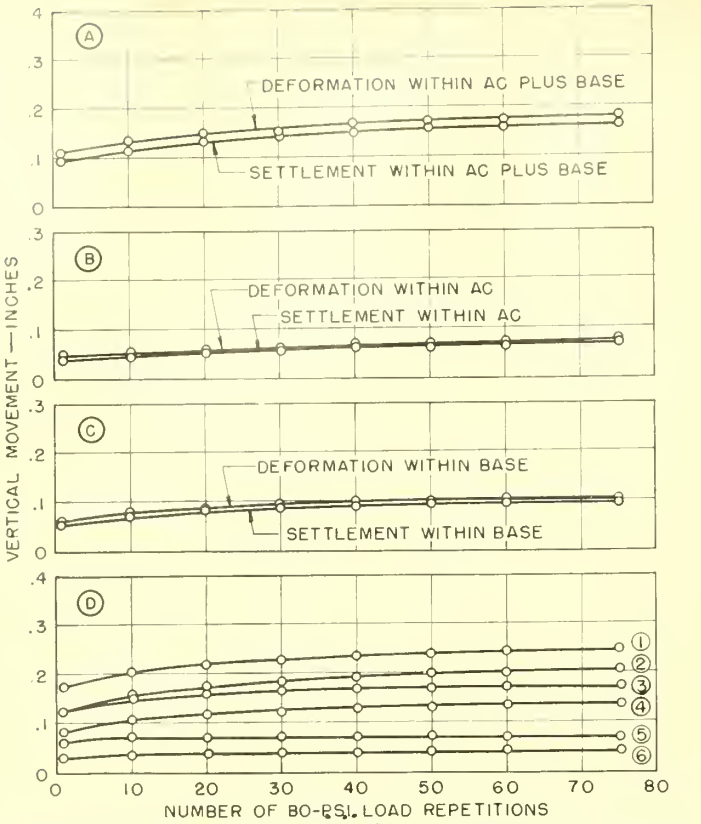

A
DEFORMATION WITHIN AC PLUS BASE
SETTLEMENT WITHIN AC PLUS BASE
B
DEFORMATION WITHIN AC
SETTLEMENT WITHIN AC
C
DEFORMATION WITHIN BASE
SETTLEMENT WITHIN BASE
D
VERTICAL MOVEMENT — INCHES
NUMBER OF 80-PSI. LOAD REPETITIONS
① – GROSS DEFLECTION, TOP OF AC
② – SETTLEMENT, TOP OF AC
③ – GROSS DEFLECTION, TOP OF BASE
④ – SETTLEMENT, TOP OF BASE
⑤ – GROSS DEFLECTION, TOP OF SUBGRADE
⑥ – SETTLEMENT, TOP OF SUBGRADE

Figure 12.—Typical Repetitional Test data for tests in which the movement of the subgrade was nonprogressive.

on pavement structures having overall thic nesses respectively of 15, 21, and 27 inch For one series of tests the pavement structu consisted of a 9-inch AC surface plus base a for another, the structure consisted of a 3-in AC surface plus base. The data shown these figures are tabulated in column 9 table 4 of reference (*3*). The curves sho in figure 8 are in agreement with those sho in figures 2–4 in that the base course alo tended to protect the subgrade from mov ment more effectively than an equal thic ness of 6-inch AC surface plus base. For te on the 12-inch thickness of structure, and f those made with the 18-inch plate on t 18-inch structure, the difference in unit lo relations for tests on the AC surface pl base and on the base course alone was a preciable, otherwise, differences were i significant.

Data for the 15- and 27-inch thicknesses pavement, in figure 9, show that the co bination of 9-inch AC surface plus base co sistently supported more load for a giv subgrade deflection than the 3-inch AC surfa plus base. For the 21-inch thickness pavement, very little difference was noted the unit load supported by the two designs pavement. No explanation was found f this apparent inconsistency.

A summary of data from the Accelerat Test made with the 18-inch bearing plate presented in figure 10. Total loads are sho for (1) elastic deflection of the subgrade 0.08 of an inch, (2) elastic deflection of t pavement structure of 0.16 of an inch, a (3) gross deflection of the structure of 0.40 an inch. The loads for each of three arb trarily selected thicknesses of structure—1 15, and 25 inches—were obtained direct from load-thickness plots of the data. T following comments pertain to these data:

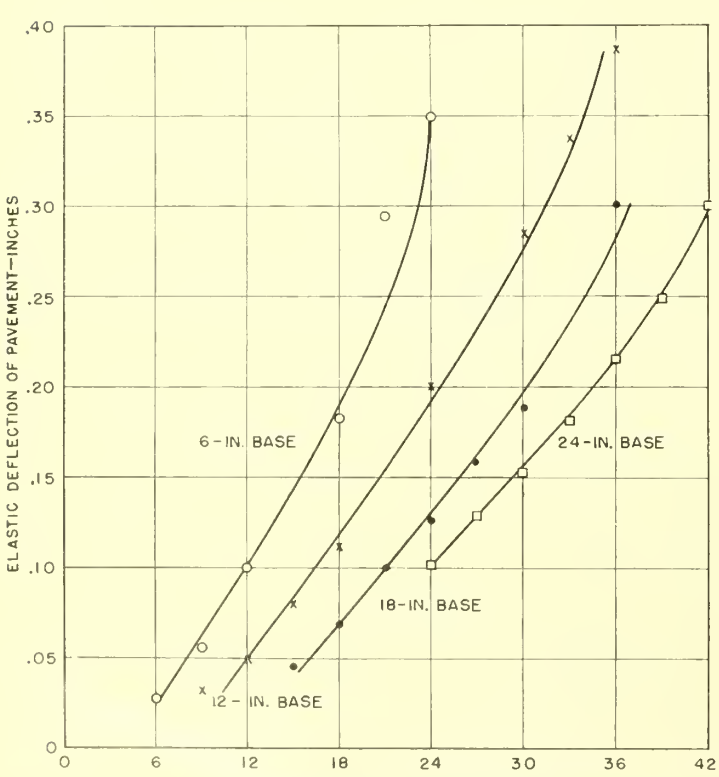


Figure 13.—Effect of size of bearing plate on elastic deflection of pavement—3-inch AC surface plus base.

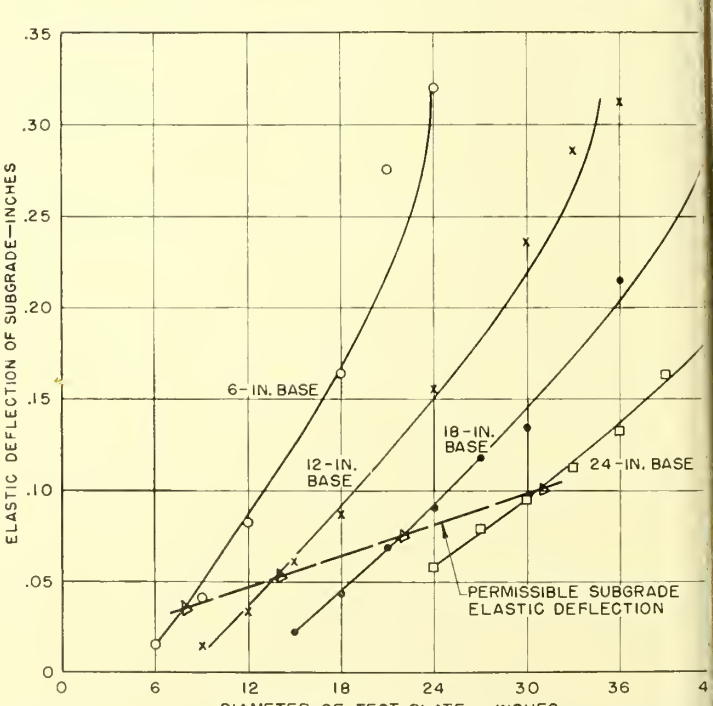


Figure 14.—Effect of size of bearing plate on elastic deflection of subgrade—3-inch AC surface plus base.

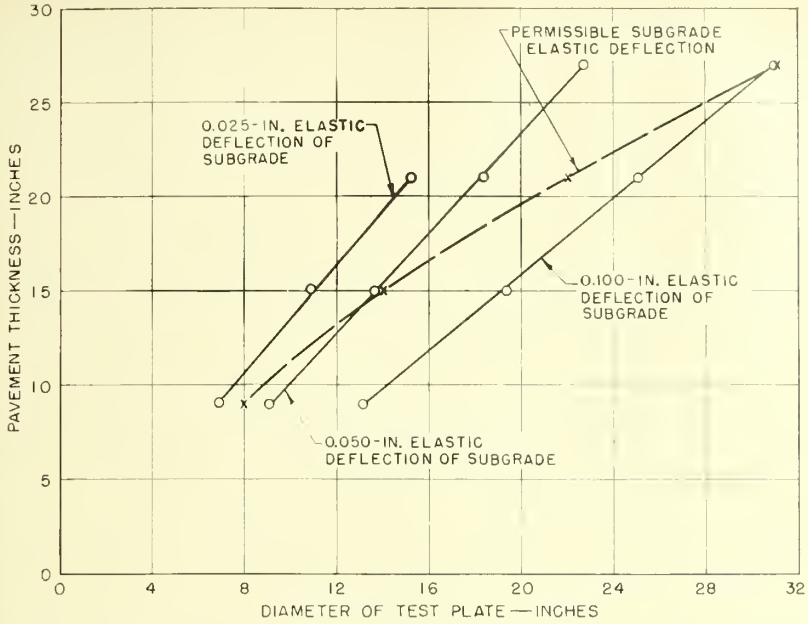


Figure 15.—Relation between size of bearing plate and pavement thickness based on elastic deflection of the subgrade—3-inch AC surface plus base.

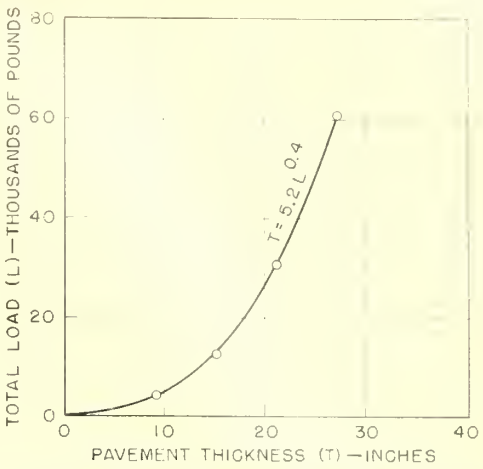


Figure 16.—Load-pavement thickness curve based on permissible elastic deflection of subgrade—3-inch AC surface plus base.

• In general, the indications are much the ıme regardless of the movement criterion sed to express load support. For example, or all three criteria the difference between the ads supported by the base course and those ıpported by the AC surface plus base course ombinations was of about the same order of agnitude. Also, the relative effect of overall ickness of pavement was about the same gardless of the movement criterion used.

• The total loads supported by a given ickness of base course alone increased some s the thickness of AC surface removed was ıcreased from 3 to 6 inches. However, an ıcrease from 6 to 9 inches in the thickness of C surface removed had little effect on the otal load supported by a given thickness of ase course alone, except for pavements aving overall thicknesses of 15 and 25 ıches when the gross deflection of the pave- ıent was 0.40 of an inch.

• Increasing the thickness of surface from to 6 inches apparently was of more benefit ıan increasing it from 6 to 9 inches. This pplied for all three thicknesses of pavement ructure. The results of the Accelerated est will be discussed further in connection ith the analysis of the Repetitional Test ata.

Analysis of Repetitional Test Data

The immediate objective in each of the epetitional Tests was to determine the ıagnitude of the load under which the pave- ıent or its supporting subgrade would act in n essentially elastic manner when a constant nit load was applied repetitiously on the avement surface. It was reasoned that, if n elastic condition were developed a progres- ive permanent deformation of the subgrade ould not occur and continued satisfactory tructural performance of the pavement might be expected. In other words, the maximum load under which the desired degree of elastic action developed might constitute the maximum safe load for a particular pavement structure.

Some of the data developed from the Repetitional Tests, part *b*, are shown in figures 11 and 12. These data are typical of those obtained from each of these tests. In figure 11 the data are representative of those obtained when some progressive vertical movement of each component of the pavement structure and the subgrade occurred during the 75 applications of load. Figure 12 shows curves for a combination of load and pavement design conditions during which the subgrade had almost no progressive movement after about 10 applications of load. The data from these two tests, in a sense, represent the two extremes of the results obtained—in the former the movements of all components were progressive, but in the latter, the movement of the subgrade was essentially nonprogressive. In block A, the top block respectively in each of the figures, the deformation and settlement within the AC surface plus base are shown. The deformation was the total decrease in thickness of the two courses as a given load repetition was being applied. The settlement was the permanent decrease that remained when the load was released. The difference between the two amounts of decrease has been termed the elastic deflection within the pavement.

Elastic behavior

The deformation and settlement that occurred within the bituminous surface (AC) and within the granular base course are shown in blocks B and C, respectively (figs. 11 and 12). The gross deflections and settlements of the top of the AC surface, top of base, and top of subgrade occurring during these tests are shown in blocks D. Results from the Repetitional Tests showed that the elastic deflection (the difference between the gross deflection and settlement) was constant throughout the period of application and release of the 75 test loads for each of the Repetitional Tests. This relationship is illustrated by the curves in blocks D in figures 11 and 12. Note that the gross deflection and settlement relations for the combination of components—surface, base, and subgrade indicated by curves 1 and 2, the base and subgrade indicated by curves 3 and 4, and the subgrade alone indicated by curves 5 and 6—are essentially parallel. Little difference was noticed in this relationship regardless of whether the load applied through a particular plate had caused progressive, permanent deformations of the components.

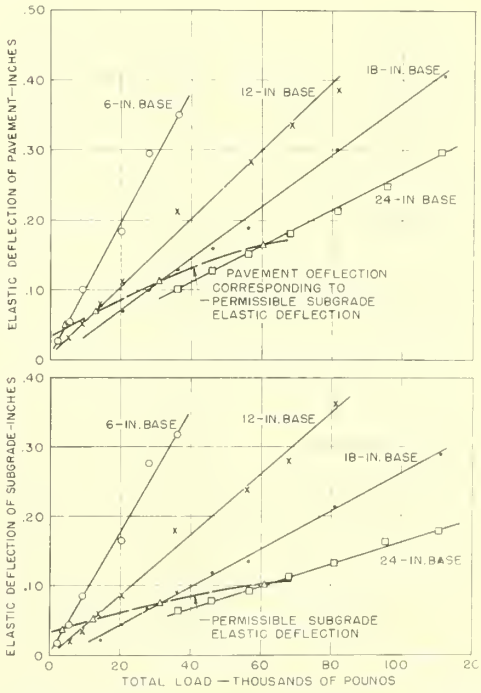


Figure 17.—Load-deflection curves for the 80-p.s.i. test load on the 3-inch AC surface plus base.

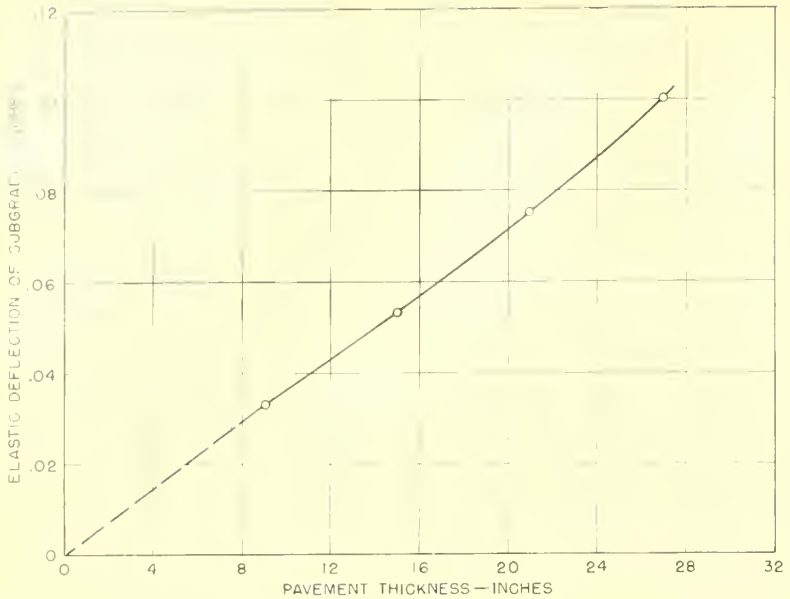


Figure 18.—Relation between pavement thickness and permissible elastic deflection of the subgrade for the 80-p.s.i. test load on 3-inch AC surface plus base.

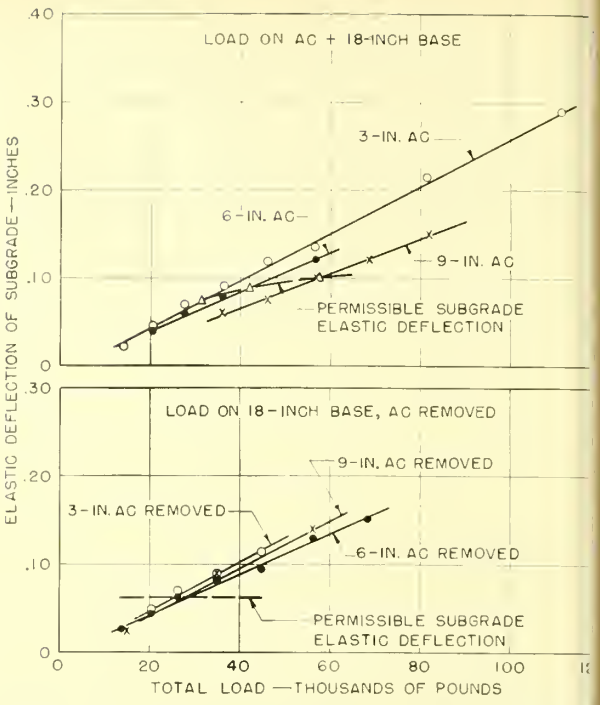


Figure 19.—Load-elastic deflection curves for surfac courses plus 18-inch base pavements and for the bas with the surface removed—80-p.s.i. test load.

Early in the Repetitional Test program, it was noticed that the subgrade appeared to act in a completely elastic manner (fig. 12) when a given thickness of overlying pavement structure protected it from progressive, permanent deformations. Consequently, a reasonable and logical assumption seemed to be that such elastic action of the subgrade might serve as an index for determining the safe load-carrying capacity of pavement. Therefore, the determination of the maximum load (80 p.s.i. times the area of bearing plate) for each test section under which the subgrade movement would cease to be progressive was adopted as another objective of this program

In all of the Repetitional Tests, none of the surface and base courses themselves ceased to undergo some progressive deformation during the application and release of the 75 loads. The extent to which the pavement plus base was permanently deformed depended, of course, upon the load-pavement design relationship. For example, according to the data shown in figure 11, the surface plus base was deformed permanently about 0.25 of an inch after 75 applications of the test load (see lower curve in block A). The corresponding permanent deformation shown in figure 12 was 0.17 of an inch. Whether the permanent deformations that developed in the surface and base were the result of consolidation or distortion, or a combination of the two, is not definitely known. Initially, it was believed that such deformations in the granular base were the result of consolidation. However, little or no evidence in support of this belief was obtained from studies of the density of the base course before and after its being loaded

Bearing plate relation to elastic deflection

Influence of the size of bearing plate on the elastic deflection produced by the 80-p.s.i. test load applied on the pavement surface is shown in figures 13 and 14. Data shown in these figures were collected from part *b* of the Repetitional Test for the different pavement sections consisting of 3-inch AC surfaces plus base and have been tabulated in column 7 of table 8a of reference (*3*). Figure 13 illustrates findings for the pavement structure and figure 14 shows data for the subgrade. The elastic deflections plotted in each of these figures are the averages of two or more test results. Also, for each test the deflections are averages of those for the 10th, 40th, and 75th load repetitions. Each plotted point therefore represents the average of a relatively large number of measurements. Consequently, the relations established are better than would have been shown if the elastic deflection at some arbitrarily selected repetition of the load had been utilized. The elastic deflections tended to be somewhat erratic during about the first 10 applications of the test load, after which the deflections remained practically constant.

For elastic deflections of the pavement and of the subgrade (figs. 13 and 14) of less than about 0.20 and 0.15 of an inch, respectively, the relations were well-defined and orderly. If larger elastic deflections were ignored, straightline curves could logically be drawn. Thus, within reasonable limits of total load applied to the surface of the pavement, the elastic deflections of the entire structure or that of the subgrade alone varied as a near-linear function of the diameter of the bearing plate. But it was evident that the total applied load (80 p.s.i. times area of the plate) caused progressive increases in elastic deflec tions of both the pavement and subgrad when larger bearing plates were used fo tests on a pavement structure of a give thickness. This is shown in figures 13 an 14 by the progressive increase in curvatu caused by increased load (larger plates).

Figures 11 and 12 represent the tw extremes of data obtained from the Repet tional Tests. Figure 11 shows that som residual movement of the material occurre each time the load was applied and release And figure 12 shows that the subgrade soi after about the 10th application of loa acted in an elastic manner; that is, it deflecte and recovered completely as each load wa applied on the surface of the pavement an then released. From similar plots prepare for each of the tests made on each of th pavement sections, it was possible by inte polation to determine the approximate max mum total load and corresponding plate si that did not produce progressive permanen movement of the subgrade. These dete minations and the resultant elastic deflecti of the subgrade are listed in table 3.

Table 3.—Permissible subgrade elastic de flection and corresponding total loads fo pavement sections of 3-inch AC plus ba

Pavement structure, thickness	Plate diameter	Total load, 80-p.s.i unit load	Permissible subgrade elastic deflection
Inches	*Inches*	*Pounds*	*Inches*
9	8	4,020	0.034
15	14	12,320	.054
21	22	30,400	.076
27	31	60,300	.101

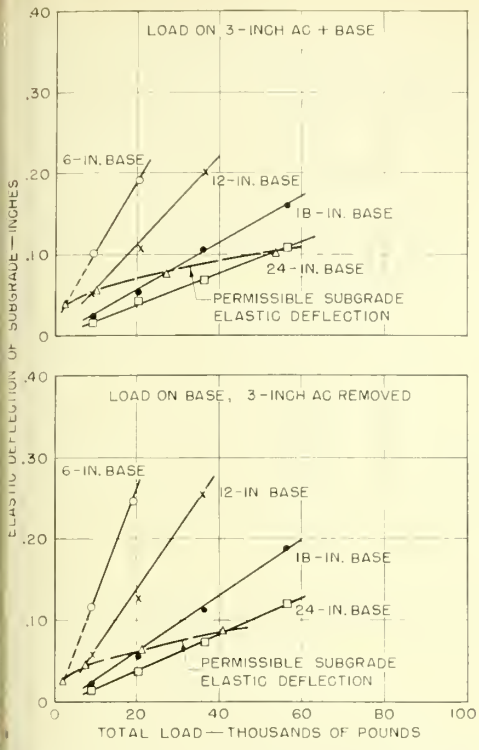


Figure 20.—Load-elastic deflection curves for 3-inch AC surface plus base pavements and for same bases with surface removed—80-p.s.i. test load—Accelerated Test.

As the subgrade elastic deflections shown in able 3 were caused by the maximum allow ible load for the selected criterion, non-progressive movement of the subgrade, they are referred to as the maximum permissible subgrade elastic deflections. The maximum permissible subgrade elastic deflections for each of the pavement sections consisting of 3-inch AC surface plus base have been plotted in figure 14. The resultant plot is a straight line. Note that as the plate size and AC-surface-plus-base thickness was increased, the permissible subgrade elastic deflection also increased. For the range in pavement structures tested (from 3-inch AC surface plus 6-inch base to 3-inch AC surface plus 24-inch base) the permissible subgrade elastic deflection increased threefold from 0.034 to 0.101 of an inch.

Load-pavement thickness relations

The relations between size of test plate and pavement thickness shown in figure 15 were developed directly from data illustrated in figure 14. The important influence of subgrade deflection on the load-thickness relation is shown clearly in figure 15. For elastic deflections of 0.025, 0.050, and 0.100 of an inch, the relations of size of bearing plate (load) to required thickness of pavement were linear. However, for the permissible subgrade deflections indicated by the intersection points of the dashed line in figure 14, the relation departed somewhat from a straight line. The load and related pavement thicknesses have been plotted in figure 16. The resultant curve shows the total load-pavement thickness relation developed by the method described; that is, it was based on non-progressive movement of the subgrade between about the 10th and the 75th repetition of an 80-p.s.i. unit load applied on the surface of the 3-inch AC-surface-plus-base pavement structure. This curve primarily illustrates the type of information that can be developed from data obtained by the Repetitional Test. The equation of the curve is

$$T = 5.2\ L^{0.4}$$

where T equals pavement structure thickness in inches, and L represents the total load in kips. The curve of load and pavement thickness is parabolic and indicates that the thickness varied as the 0.4 power of the total load.

Figure 17 shows a series of total load-deflection curves for 80-p.s.i. unit loads applied to the surface of the 3-inch AC surface plus-base sections. For data tabulations see columns 5 and 7 of table 8a in reference (*3*). The curves in the upper section of figure 17 apply to the elastic deflection of the pavement as a whole, and those in the lower section apply to that of the subgrade alone. The deflections shown are the average of those obtained from the 10th, 40th, and 75th applications of the load. These curves demonstrate that, in this type of test, the magnitude of the elastic deflection of either the entire pavement structure or of only the subgrade varied as a linear function of the total applied load. The elastic deflections of the pavement corresponding to the permissible subgrade elastic deflections are represented by the triangular symbol in the upper part of figure 17. These deflections were 0.052, 0.074, 0.112, and 0.167 of an inch for the 6-, 12-, 18-, and 24-inch bases, respectively. In the lower part of figure 17, the corresponding permissible subgrade elastic

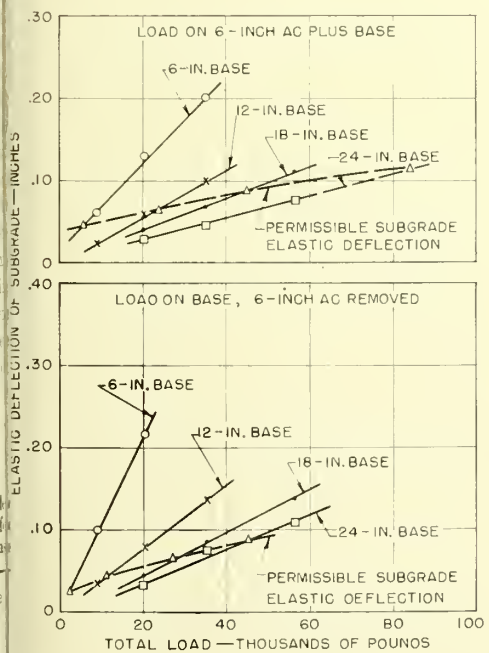


Figure 21.—Load-elastic deflection curves for 6-inch AC surface plus base pavements and for same bases with surface removed—80-p.s.i. test load—Accelerated Test.

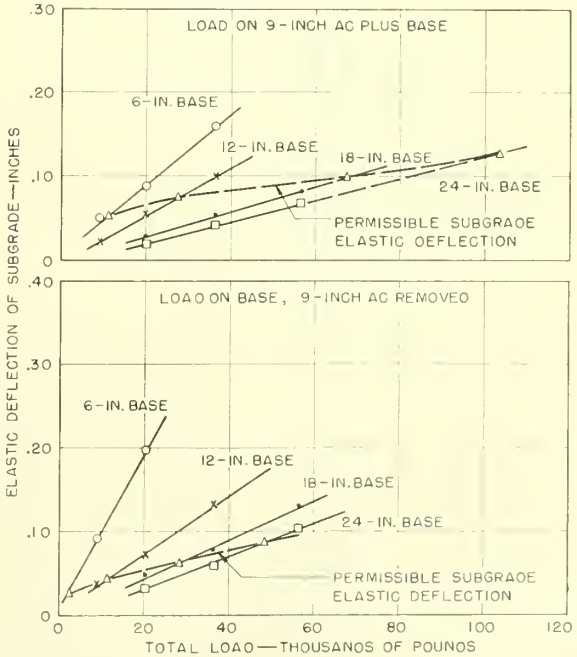


Figure 22.—Load-elastic deflection curves for 9-inch AC surface plus base pavements and for same bases with surface removed—80-p.s.i. test load—Accelerated Test.

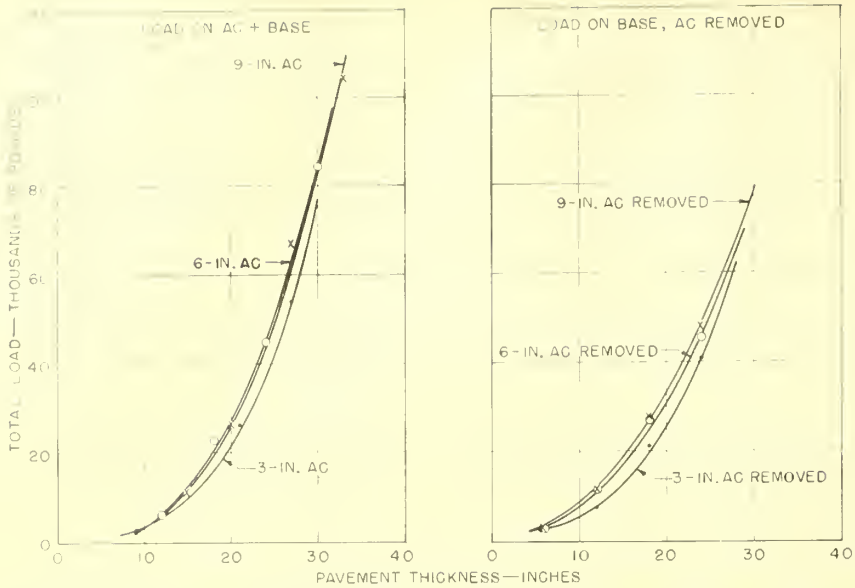


Figure 23.—Load-pavement thickness curves for surface course plus base pavements and for same bases with surface removed—80-p.s.i. test load—Accelerated Test.

deflections, 0.034, 0.054, 0.076, and 0.101 of an inch, also are indicated by the triangular symbol. The difference between each pair of deflections for a given load represents the movement that occurred within the 3-inch AC surface plus base. The lines connecting the data represented by triangular symbols in each part of the figure are somewhat curved.

In figure 18, the permissible elastic deflections of the subgrade for the 3-inch AC surface plus-base pavement have been plotted against the overall thicknesses of pavement. The permissible elastic deflection of the subgrade for any overall thickness of pavement, from 9 to 27 inches inclusive, for a unit load of 80 p.s.i. can be obtained from this plot. Although the line drawn through the plotted points has a slight curve, for practical purposes the data could be represented by a straight line drawn through the origin. This straight line could than be used for extrapolating to thicknesses of pavement less than 9 inches or more than 27 inches.

In figure 19 relations of total load to elastic subgrade deflection are shown for the three thicknesses of surfaces—3-, 6-, and 9-inch AC—on the 18-inch base course. Also shown are similar curve relations develop from tests made directly upon the 18-inc base course beneath each of the three thic nesses of surface. Again, the data we derived from part *b* of the Repetitional Tes and the tabulations are in reference (*3*).

Because of the limited number of tes made on the 6-inch and 9-inch AC surfac and directly on the 18-inch base course, was impossible to arrive at the probable sa load for these pavement sections on a basis the actual performance of the subgrade was done for the 3-inch AC pavement section However, an estimate of the load-carryin capacity of these sections can be made on th assumption that the relation between pav ment thickness and permissible elastic defle tion of the subgrade for the 3-inch AC surfac (fig. 18) applies also to the total pavemer thicknesses for the curves shown by solid lin in figure 19. The dashed line in each bloc of figure 19 was drawn through the poin established on this assumption. The ind cated loads at the points of intersection wit the load-deflection curves were 32,000, 42,00 and 58,000 pounds for the pavements consis ing of 3-inch AC surface plus base, 6-inc AC surface plus base, and the 9-inch A surface plus base, respectively. And the indicated loads were 25,000, 28,000, an 26,500, pounds for the 18-inch base beneat the 3-, 6-, and 9-inch AC surfaces, respectivel These loads are discussed more fully later this article.

In the analysis of the results of the Accele ated Tests, the different designs of pavemer were compared on a basis of the pavement ability to support load at some arbitrari selected deflection—usually the gross elastic—of the pavement and of the subgrad The development of the permissible subgrad elastic deflection from the Repetitional Tes makes it possible to reexamine the Accelerat Test data by utilizing these permissib elastic deflections as a criterion of load suppor To this end, the curves shown in figures 20, 2 and 22 were developed. Those in figure were developed from tests made on the 3-inc AC section of pavement, both upon the surfac and upon the base after removal of the surfac Those in figures 21 and 22 were developed fro tests made on the sections of pavement havin surface thicknesses of 6 and 9 inches r spectively. The elastic deflections of th subgrade for a unit load of 80 p.s.i. used in th development of the curves shown in figur 20 and 21 were obtained directly from colum 8 in table 4 of reference (*3*). The total loa are the product of the area of the bearing pla and the unit load. The dashed lines (pe missible subgrade elastic deflections) intersec the individual curves at deflections correspon ing to those shown by the curve in figure 1 for the pertinent, pavement structure thick ness. The points of intersection of the dashe lines and the load-deflection curves indicat those loads that the sections might safely hav supported under repetitive loading on rigi plates.

In figure 23, these loads have been plotted a a function of the overall thickness of pavemen The resultant curves are of the same characte

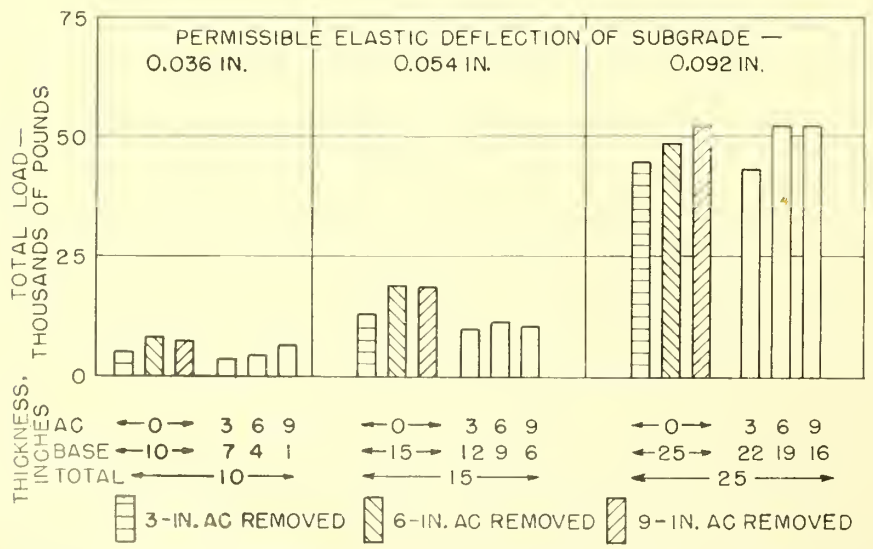


Figure 24.—Comparison of total loads supported by pavements of different thicknesses and composition—80-p.s.i. test load—Accelerated Test.

s those developed from the series of Repetitional Tests made on the 3-inch AC surface plus-base pavement (fig. 16). In fact, if the curves for these particular pavement sections were superimposed over those shown in figure 6, a remarkably close agreement would be found between them. As for the data from the Repetitional Tests, the data in figure 23 indicate that the required thickness of pavement varies as an exponential function of the applied load—approximately as its square root. This seemed to be true both where the pavement structure consisted of a range in thicknesses of surface plus base and where the pavement structure consisted of base course alone. It is emphasized that the load-pavement relations illustrated in figure 23 were based upon the assumption that the permissible elastic deflections of the subgrade corresponded to those developed from the comprehensive series of Repetitional Tests on the 3-inch AC surface plus-base pavement.

Comparison of Accelerated and Repetitional Test Data

The results of the Accelerated and the Repetitional series of load tests have been summarized in figures 24 and 25. In figure 24 the comparisons shown are of the same general nature as those shown for the Accelerated Test data in figure 10. However, the total loads shown in figure 24 were obtained from data in figure 23 for 10-, 15-, and 25-inch thicknesses of pavement structure and are for a unit load of 80 p.s.i. The permissible subgrade elastic deflections are also shown. The deflections were obtained from the relations of pavement thickness and permissible elastic deflection shown in figure 18.

The following comments pertain to the data shown in figure 24. Except for the 6- and 9-inch AC surfaces for the 25-inch total pavement thicknesses, the granular base considered as a complete structure was somewhat more effective in supporting a given load without having progressive permanent movement of the subgrade than was an equal thickness of pavement that consisted of either 3, 6, or 9 inches of surface on a granular base. The differences were consistent for the data shown in figure 24, as were the comparable differences shown in figure 10.

Comparisons

Comparisons for as many as possible of the results of the Accelerated and Repetitional tests have been shown in figure 25. To avoid undue extrapolation of the data, these comparisons were limited to pavement thicknesses of 18, 24, and 27 inches. The data showed good agreement between the results of the two series of tests when consideration is given to the fact that these tests were made a year apart and the testing procedures differed appreciably. These data indicated that the relative effectiveness of the different pavement sections to protect the subgrade was similar to that shown by data in figure 24.

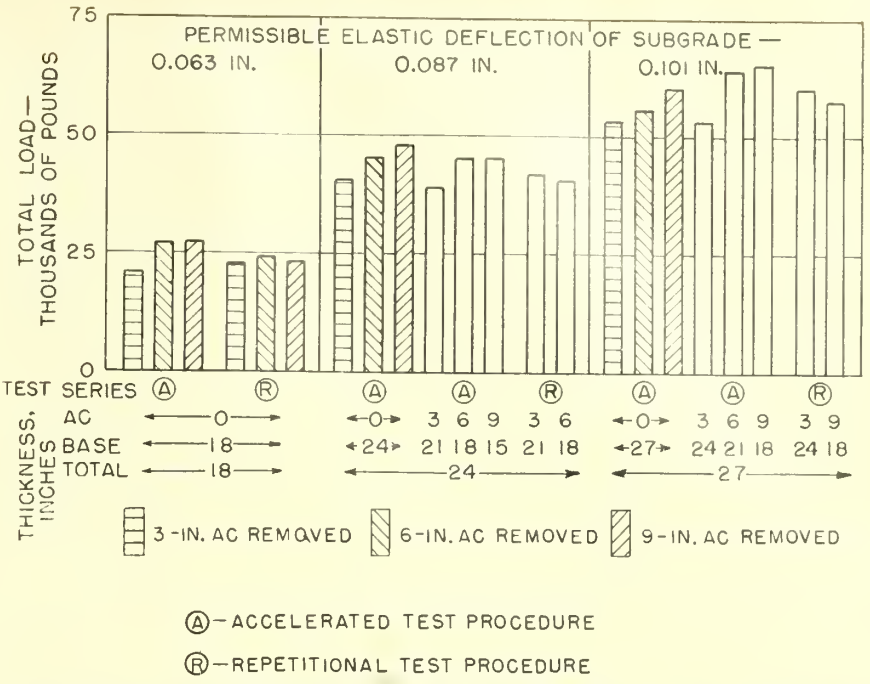

PERMISSIBLE ELASTIC DEFLECTION OF SUBGRADE —
0.063 IN.
0.087 IN.
0.101 IN.
TOTAL LOAD — THOUSANDS OF POUNDS
75
50
25
0
TEST SERIES Ⓐ Ⓡ Ⓐ Ⓐ Ⓡ Ⓐ Ⓐ Ⓡ
THICKNESS, INCHES
AC ←0→ ←0→ 3 6 9 3 6 ←0→ 3 6 9 3 9
BASE ←18→ ←24→ 21 18 15 21 18 ←27→ 24 21 18 24 18
TOTAL ←18→ ←24→ ←27→
3-IN. AC REMOVED
6-IN. AC REMOVED
9-IN. AC REMOVED
Ⓐ—ACCELERATED TEST PROCEDURE
Ⓡ—REPETITIONAL TEST PROCEDURE

Figure 25.—Comparison of total loads supported by pavements of different thicknesses and composition—80-p.s.i. test load—Accelerated and Repetitional Tests.

Elastic deflection-load relations

The elastic deflections of the different pavement components have been discussed previously. It was pointed out that the magnitude of the elastic deflection of the entire pavement structure, or that of the base plus subgrade, or that of the subgrade alone was practically constant throughout the period of application and release of the 80-p.s.i. loads. Substantiating evidence for this conclusion has been presented in figures 26 and 27 from the results of part *b* of each of the Repetitional Tests made on the 3-inch AC surface plus-base pavement. The data are tabulated in column 6 of table 8a, reference (*3*). The elastic deflections of the pavement and subgrade have been shown in figures 26 and 27, respectively, as a function of the number of load repetitions. Generally, for the smaller plates and smaller loads, the deflections were practically constant but for the larger plates and greater loads there was a tendency for the magnitude of the elastic deflection to increase somewhat during

Table 4.—Comparison of vertical movements of pavement components and subgrade

	Test conditions—							
Pavement structure	3-inch AC plus 6-inch base.		3-inch AC plus 12-inch base.		3-inch AC plus 18-inch base.		3-inch AC plus 24-inch base.	
Plate diameter [1] inches	8		14		22		31	
Total load, at 80-p.s.i. unit load kips	4.0		12.3		30.4		60.4	
ELASTIC DEFLECTION (Averages of 10th, 40th, and 75th load applications)								
Component	*Inches*	*Percent*	*Inches*	*Percent*	*Inches*	*Percent*	*Inches*	*Percent*
AC surface	0.008	16	0.008	11	0.016	14	0.039	23
Base	.010	19	.012	16	.020	18	.027	16
Subgrade	.034	65	.054	73	.076	68	.101	61
Entire pavement structure	.052	100	.074	100	.112	100	.167	100
GROSS DEFLECTION (at 75th load application)								
AC surface	0.070	22	0.099	31	0.098	28	0.112	27
Base	.152	48	.130	41	.132	37	.170	40
Subgrade	.094	30	.090	28	.122	35	.140	33
Entire pavement structure	.316	100	.319	100	.352	100	.422	100
RATIO OF ELASTIC TO GROSS DEFLECTION								
AC surface	11		8		16		35	
Base	7		9		15		16	
Subgrade	36		60		62		72	
Entire pavement structure	16		23		32		39	

[1] Plate size, obtained by interpolation, that caused nonprogressive movement of the subgrade.

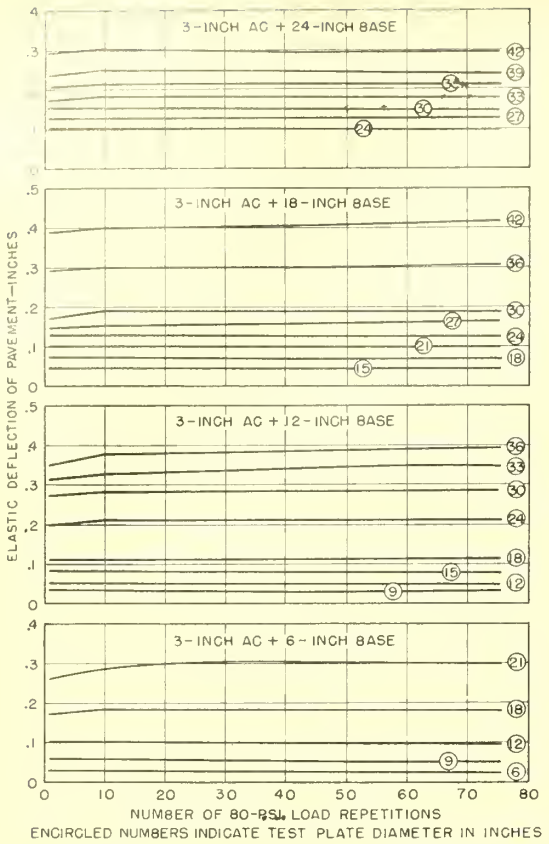

3-INCH AC + 24-INCH BASE
3-INCH AC + 18-INCH BASE
3-INCH AC + 12-INCH BASE
3-INCH AC + 6-INCH BASE
ELASTIC DEFLECTION OF PAVEMENT—INCHES
NUMBER OF 80-RSV. LOAD REPETITIONS
ENCIRCLED NUMBERS INDICATE TEST PLATE DIAMETER IN INCHES

Figure 26.—Effect of load repetitions on elastic deflection of pavement—80-p.s.i. test load.

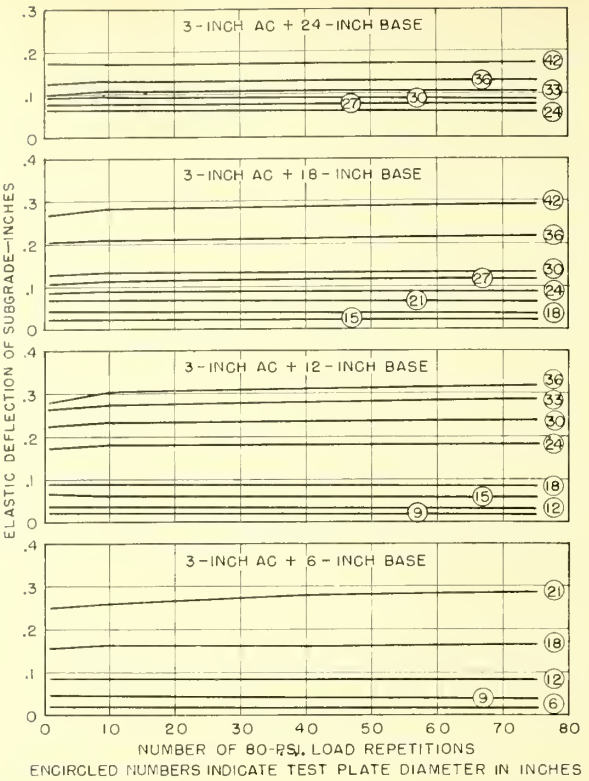

3-INCH AC + 24-INCH BASE
3-INCH AC + 18-INCH BASE
3-INCH AC + 12-INCH BASE
3-INCH AC + 6-INCH BASE
ELASTIC DEFLECTION OF SUBGRADE—INCHES
NUMBER OF 80-RSV. LOAD REPETITIONS
ENCIRCLED NUMBERS INDICATE TEST PLATE DIAMETER IN INCHES

Figure 27.—Effect of load repetitions on elastic deflection of subgrade—80-p.s.i. test load.

the first 10 repetitions of the load. With very few exceptions, the deflections remained essentially constant during succeeding repetitions of load. The differences between the deflections presented for the pavement in figure 26 and for the subgrade in figure 27 represent the magnitude of the elastic movement that occurred within the surface plus base.

Vertical movement

The method developed for measuring the vertical movements occurring at different levels in the pavement structure provided data for many ways of studying the elastic and gross deflections of the separate components. For example, in figures 28 and 29, respectively, the elastic and gross deflection measurements have been plotted against the diameters of the test plate for the entire pavement, the base plus the subgrade, and the subgrade alone. The differences between the deflections shown in each of the two upper curves in each block of the figures represent the deflections that occurred within the 3-inch AC surface. The difference between the two lower curves in each block represents the deflection that occurred within the granular base. The elastic deflections of the supporting subgrade itself are shown directly by the lowest curve.

The heavy vertical lines drawn through the three curves in each block of figures 28 and 29 designate the maximum size of plate that did not produce progressive permanent movement of the subgrade when loaded on the pavement surface. These plate sizes were obtained by interpolation or extrapolation (table 3). The relation between plate diameter and elastic and gross deflections shown by the curves in these figures indicates that, for loads applied on the pavement surface, the major portion of the elastic deflection occurred in the subgrade and that the

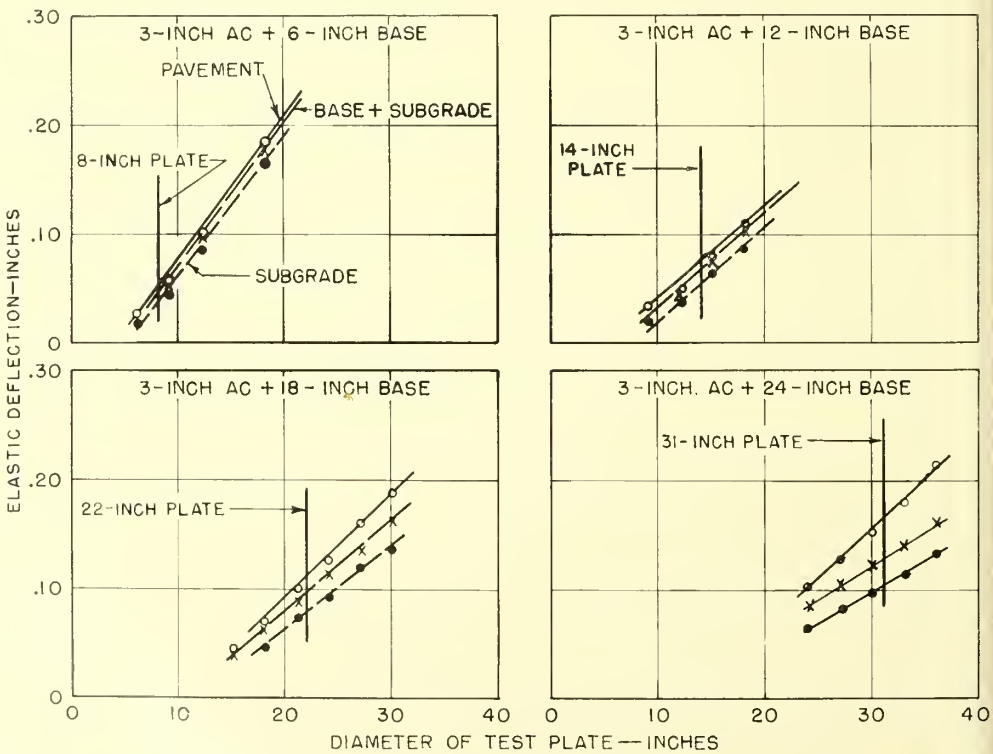


Figure 28.—Relation between size of bearing plate and elastic deflection of the pavement, the base plus subgrade, and the subgrade alone—80-p.s.i. test load.

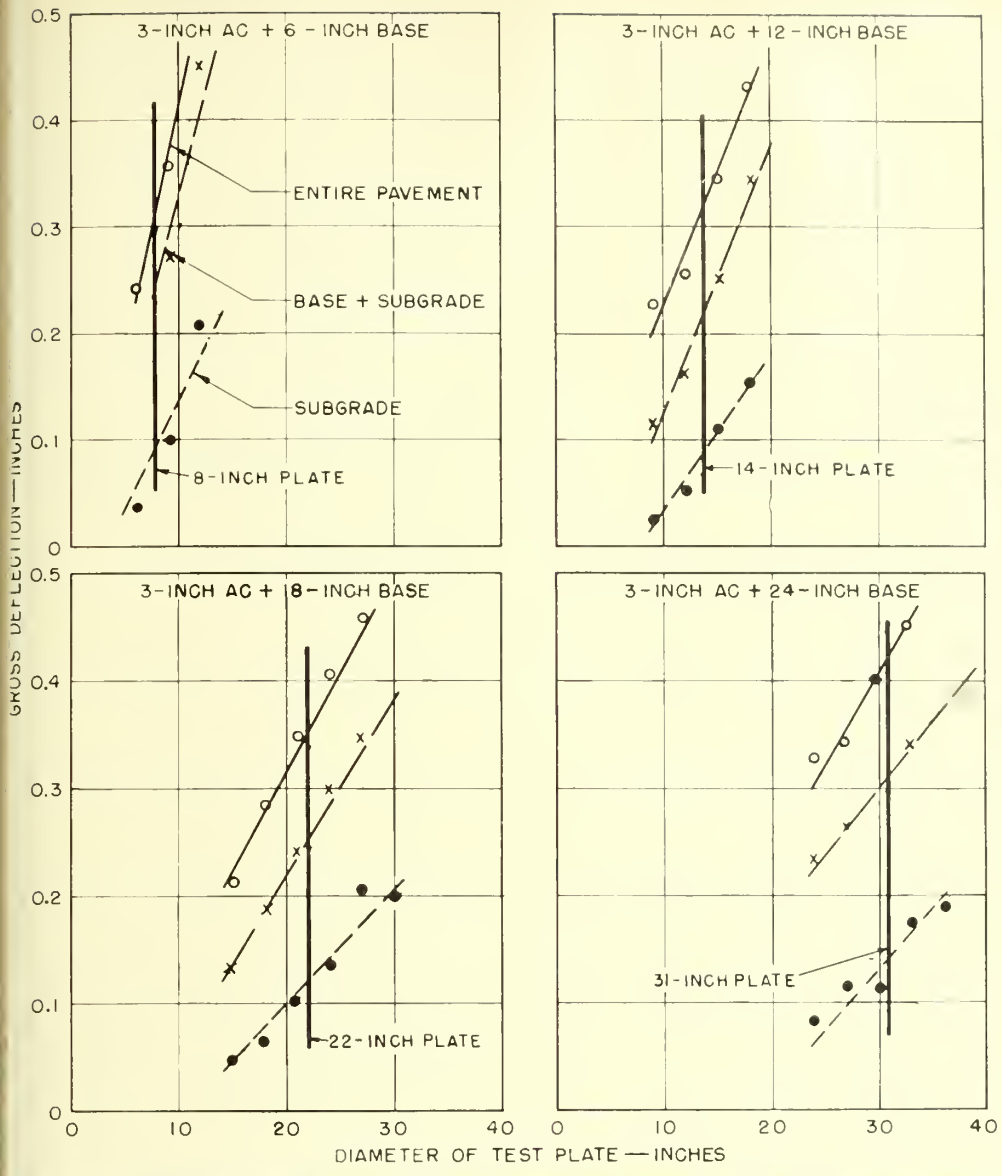


Figure 29.—Relation between size of bearing plate and gross deflection of the pavement, the base plus subgrade, and the subgrade alone—80-p.s.i. test load.

Table 5.—Effect of temperature[1] of the surface course on the load-bearing capacity of the test sections

Pavement section and time of test	Total load, thousands of pounds, for—					
	0.08-inch elastic deflection of subgrade and overall thickness of—			0.40-inch gross deflection of structure and overall thickness of—		
	10 inches	15 inches	25 inches	10 inches	15 inches	25 inches
Base course, beneath 3-inch AC:						
Spring	17.6	25.8	50.2	28.9	39.4	64.2
Summer	14.5	21.7	43.0	29.0	39.4	64.2
3-inch AC plus base:						
Spring	19.9	28.5	52.0	33.9	43.9	67.9
Summer	12.7	19.0	37.1	28.1	36.2	57.5
6-inch AC plus base:						
Spring	23.5	31.7	54.3	36.2	45.7	67.9
Summer	17.6	24.4	45.2	28.5	37.6	62.0
9-inch AC plus base:						
Spring	24.9	34.4	57.0	36.2	45.7	67.9
Summer	14.9	20.4	39.4	26.7	33.9	53.4

[1] Approximate temperature range of AC surface:
Spring, 40° to 50° F.
Summer, 75° to 95° F.

deflection of the different component combinations tended to vary as a linear function of the diameter of the bearing plate for the range in plate sizes shown. For the pavements with 6- and 12-inch bases (upper blocks in figs. 28 and 29) the curves, representing movement of the entire pavement structure and of the base plus subgrade, are practically parallel. This indicates that the movements within the AC surface remained constant as the plate size (load) was increased. However, the curves for the base plus subgrade and the subgrade alone tend to diverge as the plate size was increased, indicating that the greater the load the greater the movements within the granular base. For the 18- and 24-inch base courses all the curves tend to diverge as plate size was increased.

Data in table 4 show the elastic and gross deflections of each component of the pavement and of the entire pavement structure. The portion of the total movement that occurred in each component, expressed in percentages, is also shown. The data listed are for the four plates—8, 14, 22, and 31 inches in diameter—that were selected as representing the maximum size through which 80-p.s.i. loads could be applied to the pavement surface without causing progressive permanent movement in the subgrade. The ratios of the elastic deflection to the gross deflection, expressed in percentages, are also listed in table 4. These data were obtained from that shown by figures 28 and 29. The following comments apply to these data.

- Most of the elastic deflection occurred in the subgrade. About the same portion, 70 percent, occurred in the subgrade for each of the tests made with the four combinations of load and design for which results were tabulated. About half of the remaining percentage of deflections occurred in the AC surface and half in the base.
- The gross deflection occurring in each of the components was more evenly divided than the elastic deflection; about 30 percent occurred in both the AC surface and the subgrade, and about 40 percent in the base. These percentages were about the same for each of the load-design combinations.
- The ratios of elastic to gross deflection indicate that by far the major portion of the total vertical movement was permanent. For the 6-inch base section, only 11 percent of the movement occurring within the 3-inch AC surface was elastic and only 7 percent occurring within the base was elastic. For the 24-inch base section, the corresponding movement percentages were appreciably greater, 35 and 16 percent, respectively. The behavior of the subgrade was markedly more elastic than that of the AC surface and base, the range in ratios of elastic to gross deflection being 36 to 72 percent.

MISCELLANEOUS TESTS

Effect of Bearing Plate Size Under Constant Total Load

In connection with the program of Repetitional Tests a limited study was made of the effect that the size of bearing plate had on the

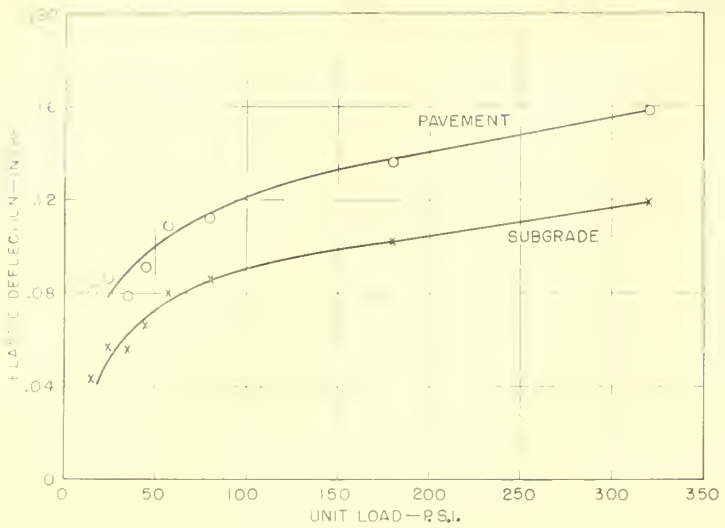


Figure 30.—Relation between unit load and elastic deflection for a total load of 20,320 pounds on bearing plates 9 to 42 inches in diameter—3-inch AC surface plus 12-inch base.

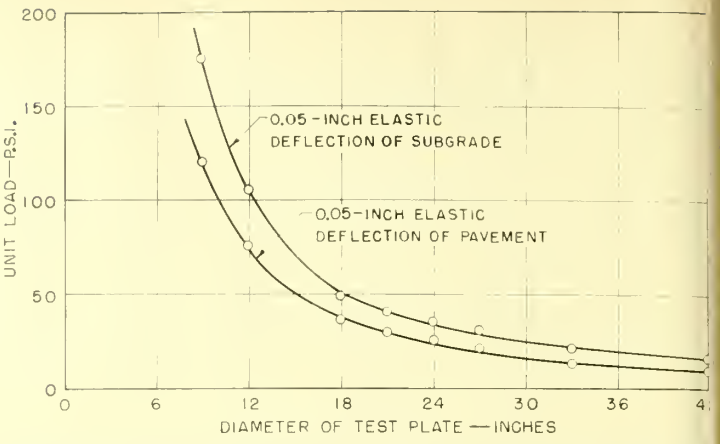


Figure 31.—Relation between unit load and size of bearing plate for 0.05-inch elastic deflection of the pavement and subgrade—3-inch AC surface plus 12-inch base.

pavement structure when the repetitiously applied total load was held constant. The tests were confined to the 3-inch AC surface plus 12-inch base section. As for the regular series of Repetitional Tests, the plates ranged in size from 9 to 42 inches in diameter and the unit load was 80 p.s.i. The total load was 20,320 pounds, which is the equivalent of an 80-p.s.i. load applied to a plate 18 inches in diameter.

The data obtained from these tests are presented in columns 5 and 7 of table 8b of reference (*3*) and are shown here in figures 30 and 31. In figure 30, unit loads have been plotted against the elastic deflections (average of 10th, 40th, and 75th load repetition) of the entire pavement structure, and of the subgrade alone. In figure 31, the unit loads have been plotted as a function of the diameter of the test plate for 0.05 of an inch elastic deflection of the subgrade and 0.05 of an inch elastic deflection of the pavement structure.

In figure 30, the test plate diameters and corresponding unit loads for the two extreme pairs of data (the points plotted at the ends of the curves) shown respectively are 42 inches and 15 p.s.i., and 9 inches and 320 p.s.i. The curves show that, for a 20,320-pound total load, an increase in unit load from 15 p.s.i. to 100 p.s.i. caused a more marked increase in elastic deflection than an increase from 100 to 320 p.s.i. The unit load-plate size curves in figure 31 show clearly the importance of the size of test plate, particularly the smaller plates. The unit loads decreased at a decreasing rate as the diameter of the plate was increased up to 42 inches.

Effect of Temperature of AC Surface

As previously stated, in studying the ability of pavements of the flexible type to support load applied through rigid plates, it was necessary to control the temperature of the bituminous surface, particularly when the different test sections were made up of several different thicknesses of the surfacing material. Information on the effects of temperature of the bituminous surface was obtained from several series of special tests. One study was made during the spring months. Using the 12- and 24-inch diameter plates, load tests were made on the 3-, 6-, and 9-inch AC surfaces of the 6-, 12-, and 24-inch base sections and also directly on the base course of these three sections beneath the 3-inch AC surface. The procedure used for loading was similar to that used in part *a* of the Accelerated Test; that is, application and release was made of three separate load increments, each one time only. The data from these tests are presented in table 6 of reference (*3*). Comparisons of the data obtained in the spring tests with those obtained from similar tests made during the summer, table 4, reference (*3*) have been presented in table 5 and in figure 32.

The results on the effect of temperatures shown in table 5 are for those obtained in tests made only with the 24-inch diameter plate. Total loads are shown for three overall thicknesses of pavement, for the base beneath the 3-inch AC surface, and for the 3-, 6-, and 9-inch AC surface plus base for the spring and summer testing periods. Comparisons of the data obtained during the two periods of testing are shown by figure 32. Unit loads for the two periods have been plotted for the 12- and 24-inch diameter plates. The deviation of the points from the line of equality shows the extent to which the loads differed. The following comments concern the data obtained in these tests.

- On the basis of a subgrade elastic deflection of 0.08 inch, the base course beneath the 3-inch AC surface was somewhat more effective in the spring than in the summer (fig. 32). The same result can be shown when the gross deflection of the pavement structure is used as the criterion. On the basis of both elastic deflection of the subgrade and gross deflection of the pavement, the effect of the presence of the AC surface on the base course was much more pronounced in the spring than in the summer.

Effect of Surcharge

Reference was made previously to the fact that the indicated resistance to load of the base course of the various pavement sections was somewhat less beneath the thin bituminous surface than beneath the thicker ones. Initially it was believed that this was due to the weight of the overlying material or surcharge effect of the surfacing material. A limited series of preliminary tests to study the effect of surcharge was made on the base course and on the subgrade of an auxiliary test pavement; the results failed to indicate any effect of surcharge.

The matter was studied further in a subsequent series of tests made on the oval track pavement. Curves are shown in figure 33 for tests on the base course and for tests on the subgrade. The data from these tests do not appear in reference (*3*). In both cases, the unit load was related to the elastic deflection of the subgrade. In the nonsurcharge tests the overlying components were removed from an area having a diameter three times that of the bearing plate. In the surcharge tests the normal procedure for testing the base course and the subgrade was employed, that is, the opening in the pavement was just sufficient to accommodate the plate.

In the tests on the 18-inch base in which the 18-inch diameter plate was used, the curves indicate some beneficial effect of the presence of the 9-inch AC surfacing, whereas the results from the tests made with the same plate on the 24-inch base indicated no effect whatever. Little influence of surcharge was indicated by the results of tests made on the 12-inch base. In the tests on the 6-inch base, the data actually indicated that the surcharge load had a detrimental effect.

The tests on the subgrade were made with the 12- and 24-inch diameter plates beneath the pavement sections consisting of the 3-inch AC surface plus the 18-inch base. Little influence of surcharge was evident in results of tests made with the larger of the

wo plates. However, some detrimental effect of surcharge was indicted in tests nade with the 12-inch plate; these effects vere similar to the effect of tests made with he 18-inch plate on the 6-inch base beneath he 9-inch AC surface.

It was noted previously that the base ·ourses showed greater resistance when ested beneath the thick AC surfaces than hey did when tested beneath the thin AC urfaces. The results of the limited series of ionsurcharge tests did not provide an ·xplanation for this finding. In general, he data indicated no important effect of ·ureharge.

Asphaltic Concrete Surfaces Laid on Subgrade

In addition to the test sections listed in able 1 and shown in figure 1, a section 50 eet in length was constructed of 6 inches of sphaltic concrete laid directly upon the ubgrade soil at one end of the north tangent. A few plate-bearing tests were made on this ection during the same period of time the ther tests were made on the regular test ections. The results of these few tests, lthough neither consistent nor conclusive, vere considered of sufficient interest to ustify the construction of three additional ections; each section was 12 by 150 feet in ize. Asphaltic concrete 3, 6, and 12 inches hick, respectively, was placed directly on he subgrade soil in these test sections located n the south tangent.

Accelerated Tests were made on the three pecial sections of pavement. The bearing plates used were 12, 18, 24, and 30 inches in liameter. Replicate tests were made with each plate on the surface and with the 30-inch plate on the subgrade after removal of the surface. Data from these tests are presented in columns 4 and 8 of table 7 in reference (*3*) and are shown in figure 34. The averages of the unit loads for the replicate tests have been plotted as a function of the thickness of the AC surface. The three separate groups of curves apply to an elastic deflection of the subgrade of 0.08 of an inch, an elastic deflection of the pavement (AC plus subgrade) of 0.08 of an inch, and a gross pavement deflection (AC plus subgrade) of 0.40 of an inch. Also shown are the data from tests made with the 30-inch diameter plate directly on the subgrade (zero pavement thickness) beneath each of the three thicknesses of pavement. These data are tabulated in columns 1 and 5 of table 7 in reference (*3*). The following comments pertain to these data.

- Tests made with the 12- and 18-inch diameter plates indicated a definite increase in load-carrying capacity as the AC surface thickness was increased from 0 to 3, to 6, and to 12 inches.
- The curves for the 24- and 30-inch plates show that little or no increase occurred in the load supported by the thicker surfaces. In fact, sometimes a slight decrease was indicated in the unit load supported.
- The bearing capacity of the subgrade of the 12-inch AC section was about 10 p.s.i. greater than that for the 3-inch AC surface, and that of the 6-inch AC surface was between the other two.

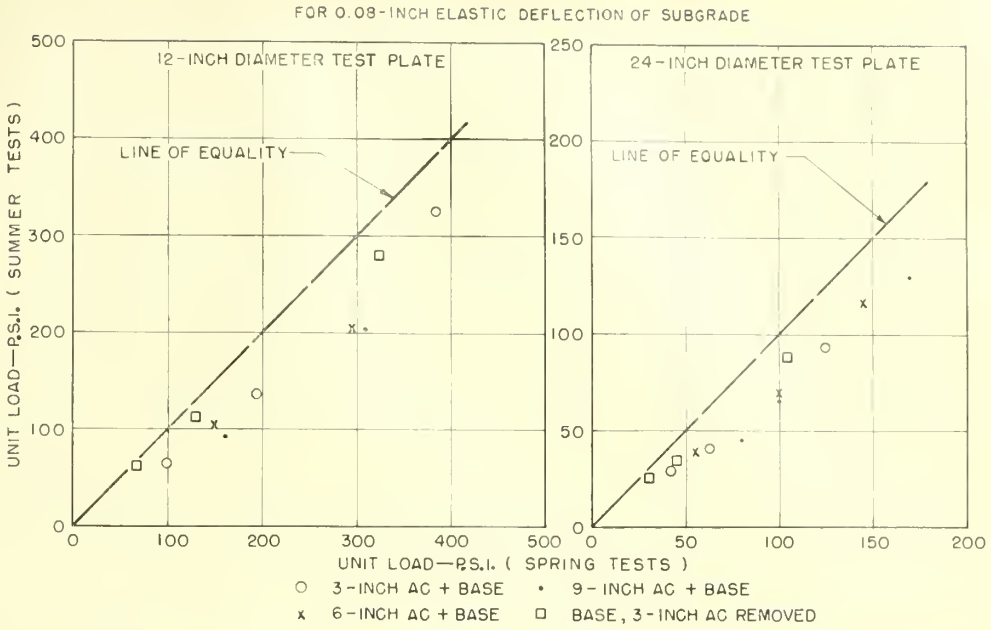

FOR 0.08-INCH ELASTIC DEFLECTION OF SUBGRADE
12-INCH DIAMETER TEST PLATE
24-INCH DIAMETER TEST PLATE
LINE OF EQUALITY
LINE OF EQUALITY
UNIT LOAD—P.S.I. (SUMMER TESTS)
UNIT LOAD—P.S.I. (SPRING TESTS)
3-INCH AC + BASE
9-INCH AC + BASE
6-INCH AC + BASE
BASE, 3-INCH AC REMOVED

Figure 32.—Comparison of load support data obtained in the spring at 40° to 50° F. with data obtained in the summer at 75° to 95° F.

Plate Bearing Tests on Subgrade

Several series of plate bearing tests were conducted to obtain a measure of the bearing capacity of the subgrade soil of both the north and the south tangents. Two procedures were employed, namely, the Incremental-Repetitional Test and the Accelerated Test. The data from the first test procedure are presented in table 3 of reference (*3*). Tests were made directly on the subgrade, after the overlying courses had been removed, as described in the following paragraphs.

- Two series of tests were made in test section 1 (table 1) with plates 12, 18, 24, and 30 inches in diameter.
- One series of tests was made in test sections 1, 2, 3, and 4 with the plate 30 inches in diameter.

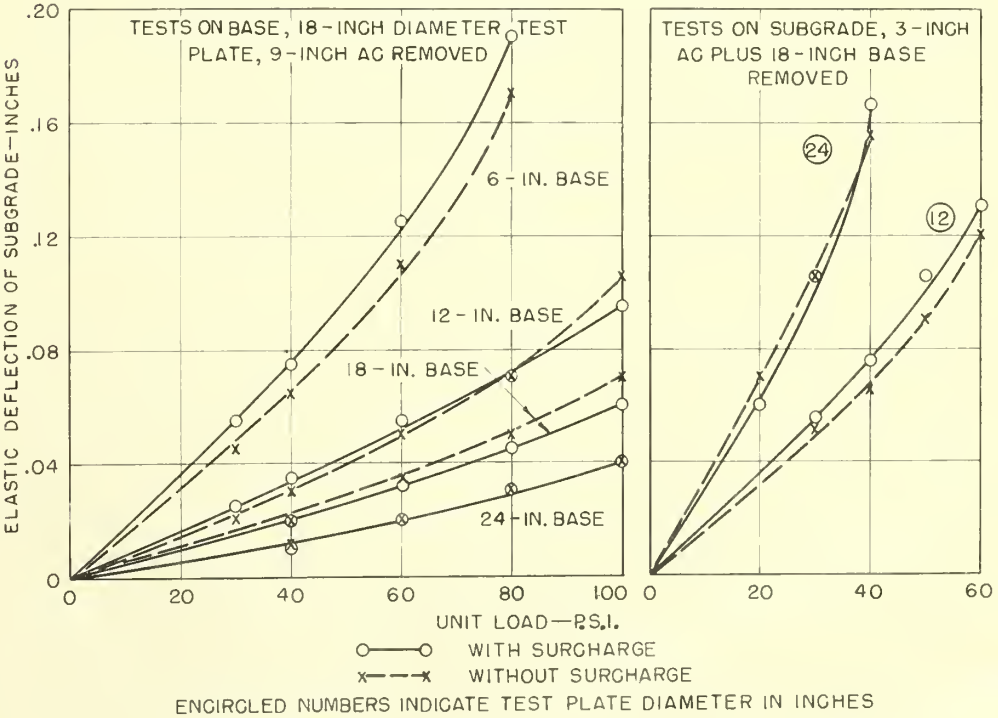


Figure 33.—Effect of surcharge based on elastic deflection of the subgrade.

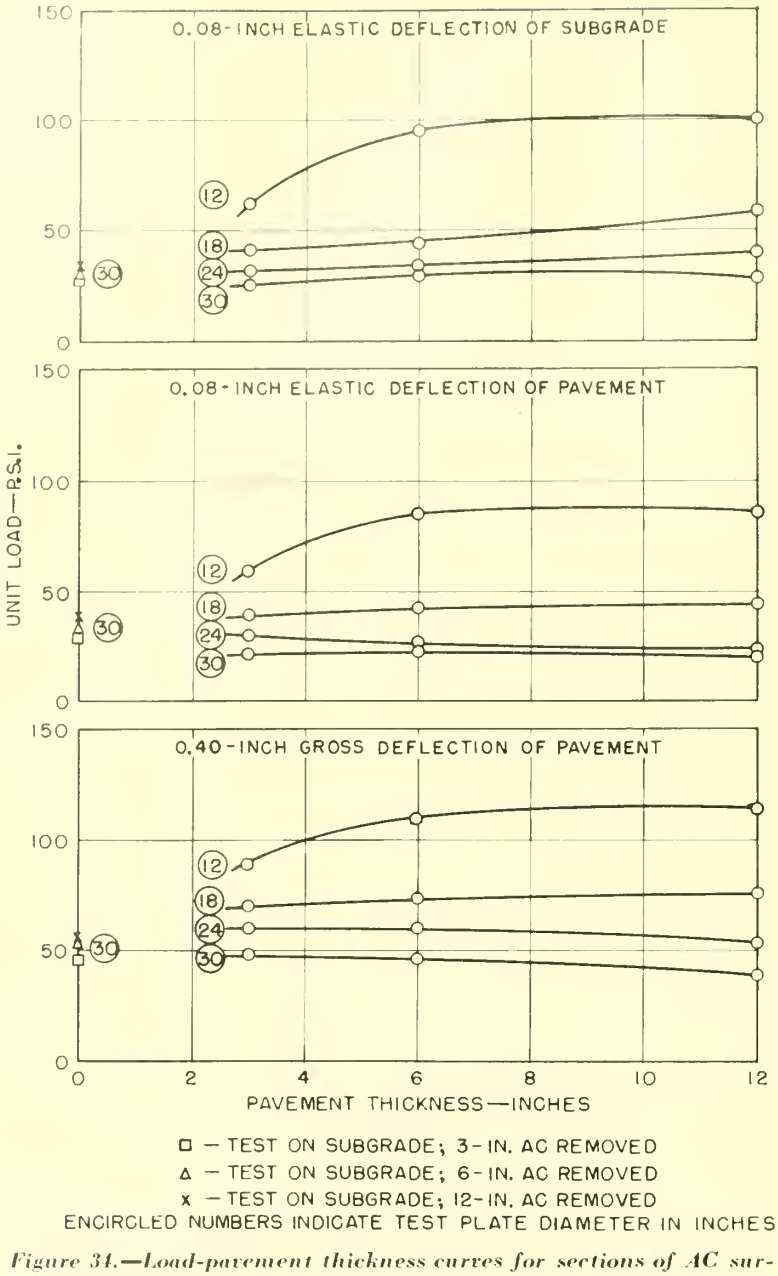

0.08-INCH ELASTIC DEFLECTION OF SUBGRADE
0.08-INCH ELASTIC DEFLECTION OF PAVEMENT
0.40-INCH GROSS DEFLECTION OF PAVEMENT
UNIT LOAD—P.S.I.
PAVEMENT THICKNESS—INCHES
□ — TEST ON SUBGRADE; 3-IN. AC REMOVED
Δ — TEST ON SUBGRADE; 6-IN. AC REMOVED
x — TEST ON SUBGRADE; 12-IN. AC REMOVED
ENCIRCLED NUMBERS INDICATE TEST PLATE DIAMETER IN INCHES

Figure 34.—Load-pavement thickness curves for sections of AC surface laid directly on the subgrade soil of the south tangent.

• One series of tests was made in test section 16 on the south tangent. Seven bearing plates whose diameters ranged from 12 to 84 inches were used. The three larger bearing plates in the group—42, 60, and 84 inches in diameter—were precast, circular concrete blocks that were 12 inches thick. The other four plates used were steel plates having diameters of 12, 18, 24, and 30 inches. The 8 inches of bank-run gravel surfacing that had previously been placed on the soil was removed before these tests were made.

The deflection data from the Accelerated Test presented in table 5 of reference (*3*) include information on one series of tests made in test section 3 with the 12-, 18-, 24-, and 30-inch plates; and for one series of tests made in test sections 1, 2, 3, and 4, with the 30-inch plate.

Curves in figures 35 and 36 were based on data from the plate load tests made on the subgrade soil beneath the pavement test sections on the north tangent. See tables 3 and 5 of reference (*3*) for tabulations. The unit load data have been plotted as a function of the diameter of the bearing plates in figure 35 and as a function of the perimeter-area ratio of the plates in figure 36. The data were plotted for both an elastic deflection of the material of 0.08 of an inch and for a gross deflection of 0.40 of an inch. From the Incremental-Repetitional Tests, the data shown for results of tests made with the 30-inch diameter plate are the average of results of 20 tests and for the other plates the average of results of three tests. For the results obtained from part *a* of the Accelerated Test, the data shown for the 30-inch plate are the average of results from 16 tests, and those shown for the other plates are averages of results from two tests.

Size of test plate

The curves in figures 35 and 36 indicate that the size of the test plate had a consistent and orderly effect on the bearing capacity of the soil. Curves in figure 35 tend to be curvilinear and those in figure 36 are linear. The difference between the relations shown by these curves and obtained by the two test procedures cannot be definitely explained. Part *a* of the Accelerated Test provided for the application and release of the three load increments one time only, and in the Incremental-Repetitional Test each of four increments was applied and released five times. This difference in procedure might have accounted for some difference in the results although perhaps not to the extent indicated by the curves. Also, the difference might partially have been caused by the fact that the Accelerated Tests were made on the subgrade beneath the 18-inch base course section, and the results of all tests made on the subgrade, as well as on the pavement of this section, provided somewhat higher loads than results of tests on the 6-, 12-, and 24-inch base course sections. Moisture and density data also failed to account for these findings.

The data from the tests made on the subgrade soil of the south tangent are shown by curves in figures 37 and 38. The nature of the curves is the same as that of curves representing results of the tests made on the subgrade of the north tangent. Based on elastic deflection, the load results were slightly higher than those obtained from tests made on the north tangent. When based on gross deflection, the load results for tests made on the two tangents were practically equal. The curves in figures 35 and 37 indicate that the ability of the soil to support load decreased at an ever diminishing rate as the size of the test plate was increased up to the maximum size used in these tests (84 inches in diameter). Linear curves in figure 38 were fitted to the load data plotted as a function of the perimeter-area ratio of the plates. Although some of the points deviated somewhat from the curves as drawn, undoubtedly a linear load-deflection relation existed.

Values based on elastic deflection were computed for the soil constant, k, from the load figures for the 30-inch diameter plate plotted in the upper part of figure 35. Soil constant values of 200 and 183 pounds per cubic inch, respectively, were computed from results of the Accelerated and Incremental-Repetitional Tests. The corresponding soil constant value for the subgrade soil of the south tangent was 208 pounds per cubic inch. Thus it can be said that the subgrade soil of the test track, when existing with a moisture content of about 26 percent and a dry density of about 100 pounds per cubic foot, had a k value of about

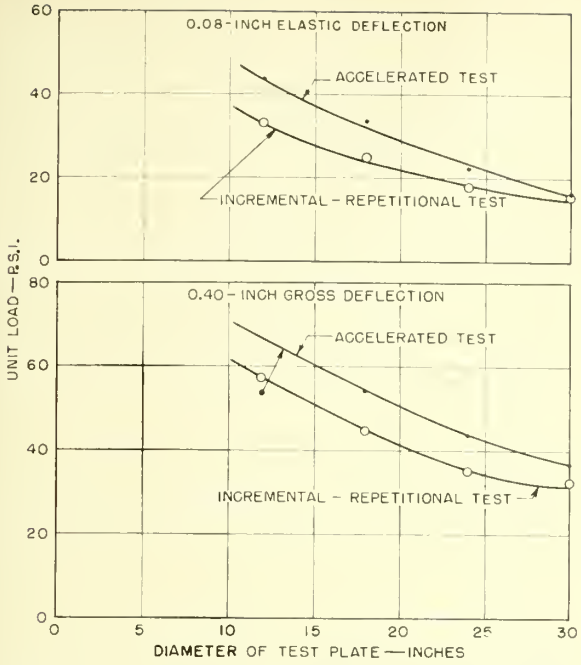


Figure 35.—Load-size of test plate curves for Incremental-Repetitional and Accelerated Tests on the north tangent subgrade soil.

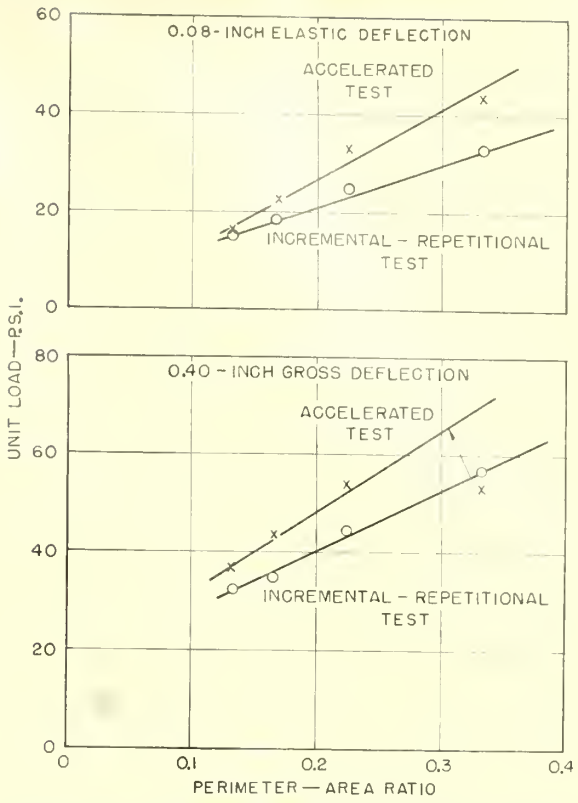


Figure 36.—Load-perimeter area ratio curves for Incremental-Repetitional and Accelerated Tests on the north tangent subgrade soil.

200 pounds per cubic inch. The laboratory CBR value of the subgrade soil was reported to be about 2 in reference (*3*).

The scheduled program of Repetitional Tests was limited to tests on the surface and base course of the pavement sections. However, enough tests of this type were made on the subgrade to determine whether, under direct loading with comparatively small unit loads, the soil would act elastically in the same manner as when the load was transmitted to it through the pavement structure. Loads varying in intensity from 5 to 20 p.s.i. were applied repetitiously through plates having diameters of 18, 30, and 42 inches, respectively.

In these Repetitional Tests on the subgrade, the elastic deflections were considerably less than the permissible elastic deflections that occurred in tests on the pavement. Nevertheless, in no instance did the subgrade soil attain a condition of completely elastic behavior during these tests. Each application of the load produced some additional permanent movement. The same type of difference in the behavior of a fine-grained soil that had been loaded directly through rigid plates and indirectly through a pavement structure has been observed previously (*7*). Differences in the elastic behavior of the soil are believed to have been caused by the manner in which the load was distributed to it. When the load was applied directly to the subgrade through a rigid plate, the highest contact pressure occurred around the peripheral area of the plate. However, when the load was applied to the subgrade through the pavement structure, the contact pressure tended to diminish outward from a maximum on the axis of loading.

Application of the Test Data

The results of the Repetitional Tests indicated that a limit exists as to the amount of repeated deflection that fine-grained subgrade soils can sustain without the pavement suffering progressive movement or deformation. Therefore, application of test data analyzed in this article to the design of new pavements would necessitate the determination of the permissible elastic deflection of the soils that would compose the subgrade. During the Hybla Valley investigation, the soil loaded directly through rigid plates never developed the same degree of elasticity that the test indicated occurred when the load was distributed through an overlying pavement structure. The design of pavements to be constructed on other types of soils would therefore require construction and testing of trial sections to determine the elastic deflection of the subgrade soil. The necessity for construction of such test sections would be obviated and data presented here would be useful if some type of load bearing test were to be developed by which the same reaction could be obtained when the subgrade soil was tested directly as when it was tested by having the load distributed through the pavement structure.

Also, if the data presented here were utilized in pavement designing, it would be necessary to ascertain whether asphalt pavements in service can tolerate without undue distress the magnitude of elastic deflections listed in table 4.

ACKNOWLEDGMENTS

The scope of the investigation at Hybla Valley required the assistance of many individuals. Contributions of engineers of the cooperating agencies and of Public Roads employees, who were intimately associated with the project, are acknowledged by the authors. W. N. Carey, Jr., represented the Highway Research Board and J. E. Driscoll the Asphalt Institute. With the exception of K. E. Young, other employees of Public Roads who made major contributions to the investigation have retired, are deceased, or are employed elsewhere. These employees were: L. W. Teller who, with the assistance of E. C. Sutherland, directed the investigation. Field testing was supervised by R. J. Lancaster and K. E. Young was an engineering aid; F. R. Olmstead directed the sampling and testing of soils and aggregates.

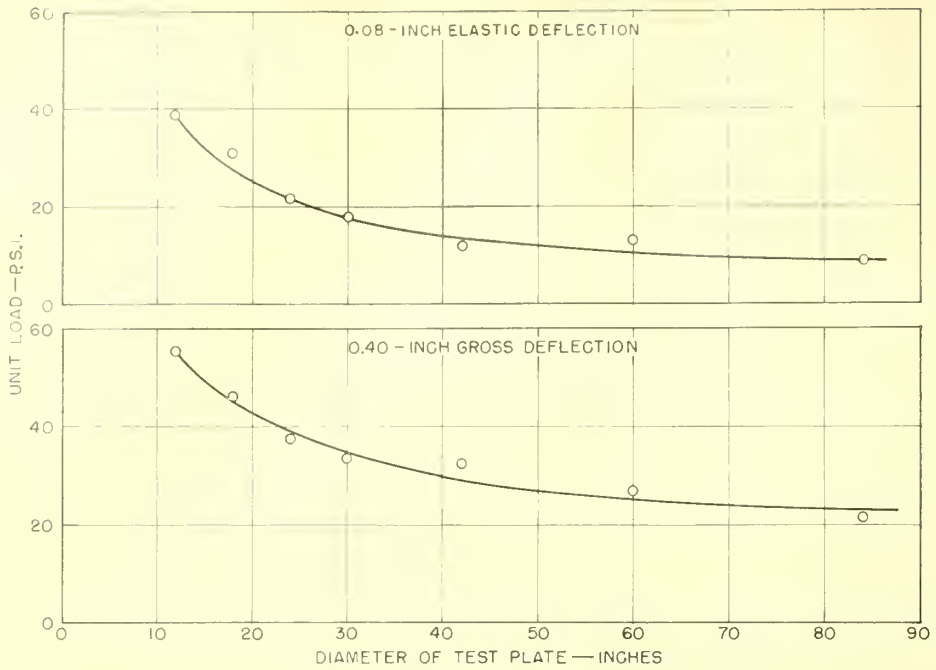


Figure 37.—Load-size of test plate curves for Incremental-Repetitional Tests on the south tangent subgrade soil.

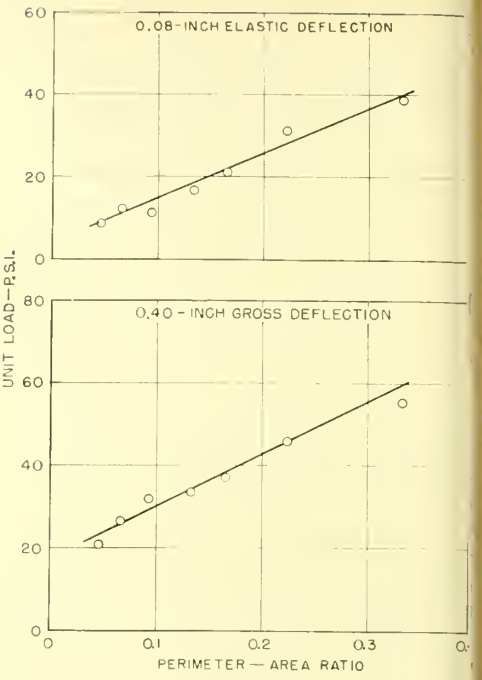


Figure 38—Load-perimeter area ratio curves for Incremental-Repetitional Tests on the south tangent subgrade soil.

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Shear Loads on Pavements

BY THE PHYSICAL RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

By [1] **EDWARD S. BARBER,**
Highway Research Engineer [2]

Stresses caused by shear loads should be considered when pavements are being designed, according to information presented in this article. Often, only the stresses caused by normal loads are considered when a stress analysis is being prepared for a pavement design. Information presented in this paper shows that stresses induced when vehicles stop may be more critical than had heretofore been realized, and that under static vehicles the effective vertical stress is increased by inward acting shear stresses caused by pneumatic tires.

Introduction

STRESS analysis of pavements is often based on normal loads alone; however, certain shear loads should not be ignored. In this article, two sources of applied shear stresses are considered—those produced by a pneumatic tire under a vertical load, and those caused by a component of the total load parallel to the surface of contact, which generally is considered to be horizontal.

Conclusions

From the information presented in this article, it is concluded that stresses caused by static loads and strong horizontal loading should be considered when a flexible pavement is being designed. An exact analysis of anticipated stresses must include consideration of these two factors, either of which causes stresses having a more critical effect on the bearing capacity of a pavement than stresses caused by a vehicle moving at constant velocity along a horizontal tangent. Under the static load exerted by pneumatic tires, the shear stresses often cause the development of a critical creep of the pavement. Critical stresses in a pavement also are greatly increased and the bearing capacity greatly decreased when accceleration or deceleration of a vehicle produces strong horizontal loading components.

Inward Acting Shear Stresses

When a vertical load is applied to a pneumatic tire on a horizontal surface, the tensile prestress in the tire carcass caused by the inflation pressure is reduced, and shear stresses acting toward the center of the loaded area are produced on the surface of contact. These shear stresses are in the opposite direction from those produced under a solid rubber tire and their magnitude may be as great as the vertical pressure; this was shown by Markwick and Starks (*1*)[3] and by Bonse and Kuhn (*2*). A shear force applied to the surface produces interior stresses in different directions, including vertical; this is shown in figure 1. The formula presented by Westergaard (*3*) is for a homogeneous isotropic elastic material.

The shear stresses on the contact area under a static pneumatic tire can be considered to be made up of rings of shear stresses. When a ring of uniform, inward acting shear stresses is considered as being made up of shear loads on infinitesimally small points, the formula in figure 1 can be integrated to give the formula shown in figure 2. Figures 3 and 4 show graphs of vertical normal stresses at several points from a ring of shear stress calculated from this formula. Because the applied shear stress has no vertical component, the total vertical normal stress at any depth caused by shear stress is zero. The total vertical normal stress is the volume between the horizontal axis and the surface that is produced by rotation of any one of the curves shown in figure 3 about the vertical axis. By use of a numerical combination of concentric rings, the vertical normal stresses produced by a uniform distribution of inward

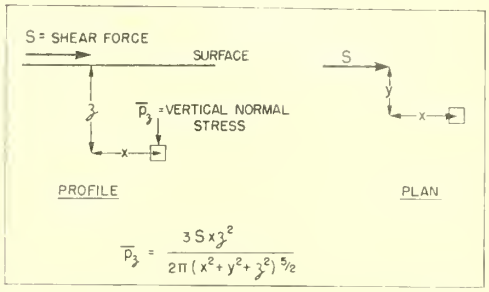

Figure 1.—Vertical normal stress from applied shear force.

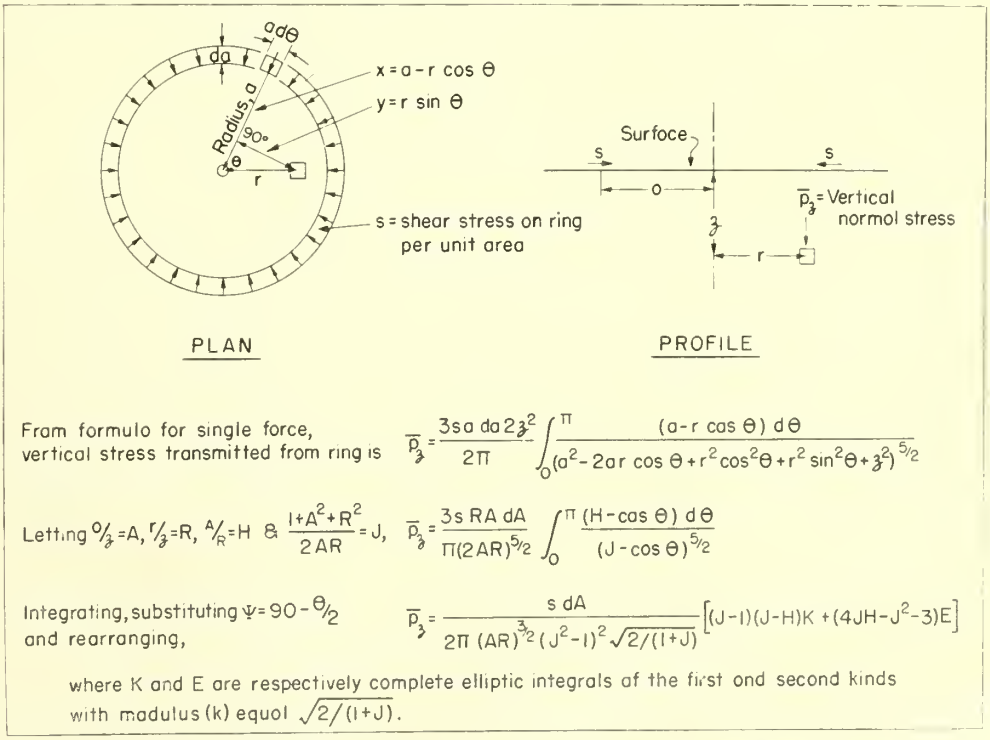
a dθ
da
x = a − r cos θ
y = r sin θ
Radius, a
90°
θ
r
s = shear stress on ring per unit area
Surface
s
s
o
$\bar{p}_z$ = Vertical normal stress
z
r
PLAN
PROFILE
From formulo for single force, vertical stress transmitted from ring is $\bar{p}_z = \frac{3sa\,da\,z^2}{2\pi} \int_0^{\pi} \frac{(a - r\cos\theta)\,d\theta}{(a^2 - 2ar\cos\theta + r^2\cos^2\theta + r^2\sin^2\theta + z^2)^{5/2}}$
Letting $^{o}/_{z} = A$, $^{r}/_{z} = R$, $^{A}/_{R} = H$ & $\frac{1+A^2+R^2}{2AR} = J$, $\bar{p}_z = \frac{3s\,RA\,dA}{\pi(2AR)^{5/2}} \int_0^{\pi} \frac{(H - \cos\theta)\,d\theta}{(J - \cos\theta)^{5/2}}$
Integrating, substituting $\Psi = 90 - \theta/2$ and rearranging, $\bar{p}_z = \frac{s\,dA}{2\pi(AR)^{3/2}(J^2-1)^2\sqrt{2/(1+J)}} \left[(J-1)(J-H)K + (4JH - J^2 - 3)E\right]$
where K and E are respectively complete elliptic integrals of the first and second kinds with modulus (k) equal $\sqrt{2/(1+J)}$.

Figure 2.—Vertical normal stress from ring of inward acting shear stress.

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[1] Presented at the International Conference on Structural Design of Asphalt Pavement, University of Michigan, Ann Arbor, Mich., August 1962.

[2] Mr. Barber is also Associate Professor of Civil Engineering at the University of Maryland, College Park, Md.

[3] The references indicated by italic numbers in parentheses are listed on page 144.

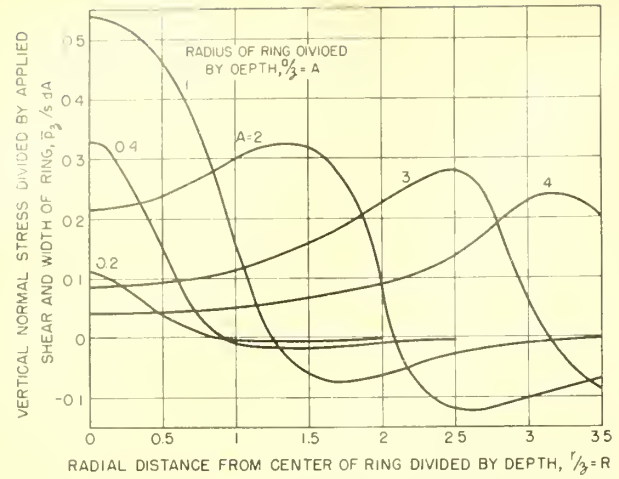
RADIUS OF RING DIVIDED BY DEPTH, r/z = A
A=2
3
4
0.4
0.2
VERTICAL NORMAL STRESS DIVIDED BY APPLIED SHEAR AND WIDTH OF RING, $\bar{p}_z/s\,dA$
RADIAL DISTANCE FROM CENTER OF RING DIVIDED BY DEPTH, r/z = R

Figure 3.—Vertical normal stress at different radial distances from shear on ring.

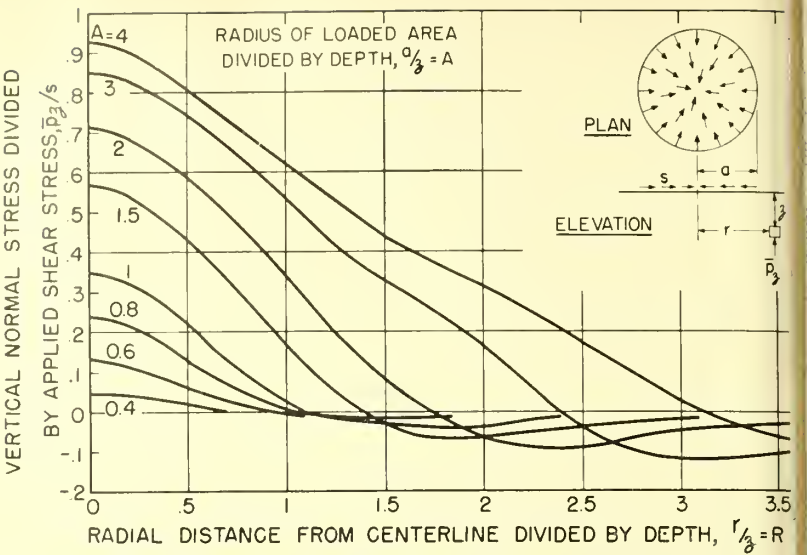

Figure 5.—Vertical normal stress from uniform inward shear over circular area.

acting shear stresses over a circular area have been calculated; these calculated vertical normal stresses are shown in figure 5.

Figure 6 shows the vertical normal stress under the center of a circular area to which several different distributions of inward acting shear stresses were applied. The parabolic distribution approximated the measured applied stresses but, for simplicity, a uniform distribution was used for this study. Stresses, measured by McMahon and Yoder (*4*) and by Foster and Fergus (*5*), under vertically loaded plates approximated those that were obtained by the application of the theory of elasticity in which only vertically applied loads were considered. The measured stresses caused by vertically loaded pneumatic tires generally were somewhat greater than those calculated for the stresses from the vertical load alone. Spangler (*6*) measured vertical pressures under a pavement that were greater than those applied at the surface. The increase in vertical stress obtained in this study when the inward acting shear stresses were included in the calculation is shown in figure 7. Therefore, the increase from inward acting shear stresses should be considered when studies are made of the effect that pavement thickness has on transmitted stresses. This concentration of vertical deflection caused by shear stresses from differen sources has been reported by Barber and Sawyer (*7*) and is illustrated in figure 8.

Shear Stresses in One Direction

Shear stresses in one direction between tir and pavement surface may be produced i several ways. Tangential forces up to abou 10 percent of vertical loads may be produced by longitudinal grades or superelevation. The ratio of tangential to normal loads is limited by the coefficient of friction between tire and surface this coefficient often reaches 0.8. This ratio occasionally is reached when wind acts on a

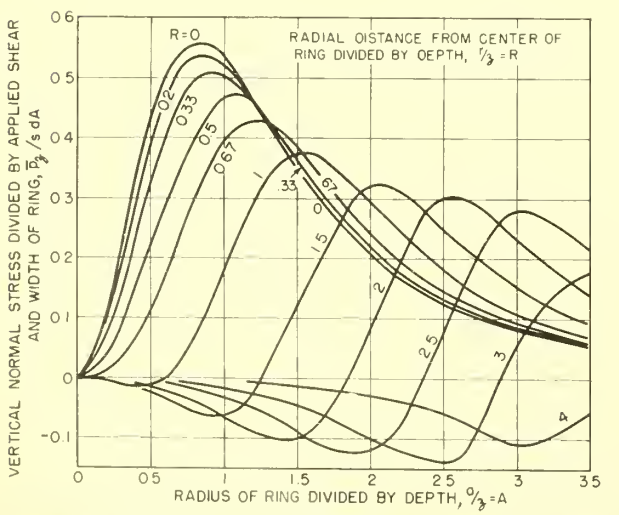

Figure 4.—Vertical normal stress from shear on rings of different radii.

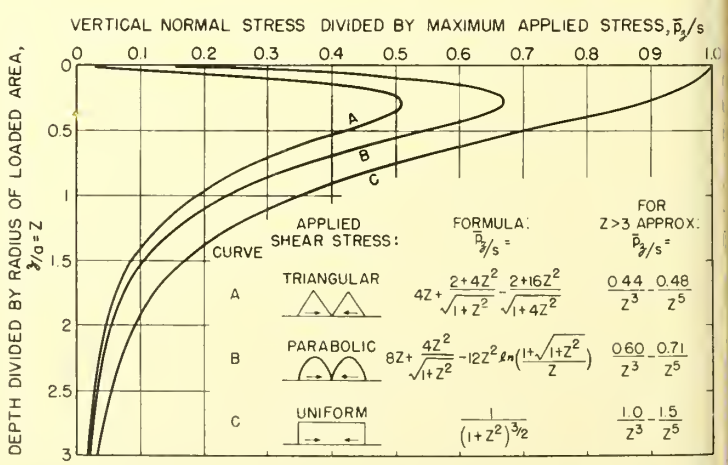

Figure 6.—Vertical normal stress under center of different inward shear stresses on circular area.

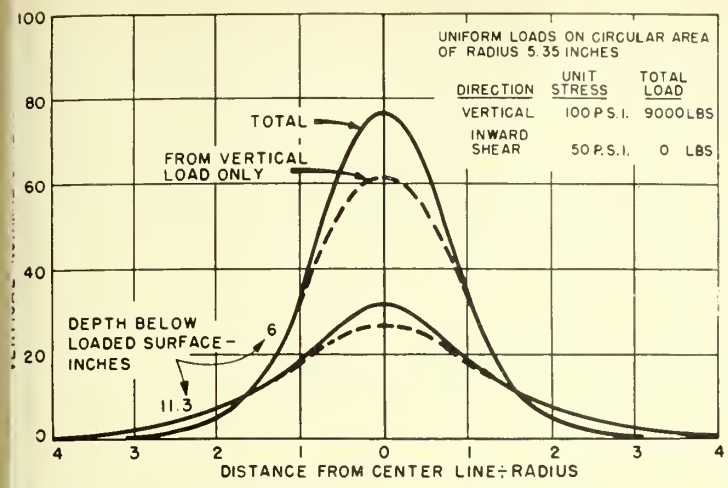

Figure 7.—Combined vertical stress from vertical and inward shear loads.

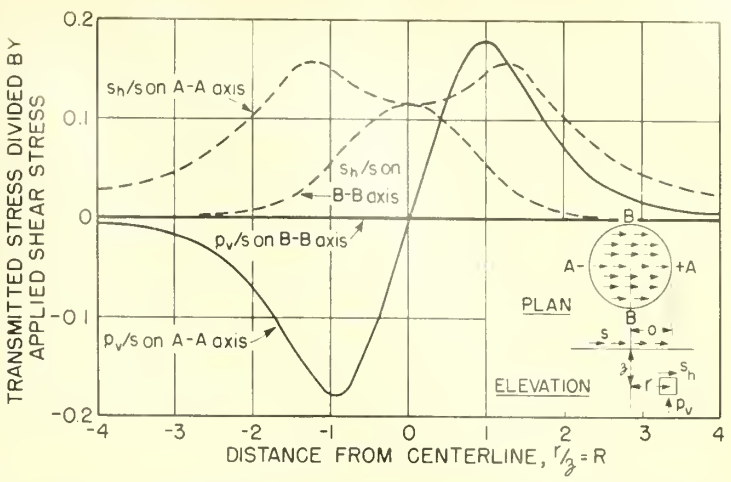

Figure 9.—Lateral distribution of stresses from shear stress in one direction at surface, A/Z=1

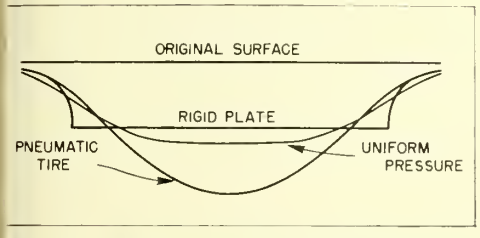

Figure 8.—Measured surface displacements of soil under different types of equal vertical loads.

ehicle, sometimes is reached when a vehicle s stopped, and may be reached when velocity f a vehicle is constant on a horizontal curve. On a curve having a 75-foot radius and no uperelevation, a vehicle traveling at a speed f 30 miles per hour develops a horizontal orce that is 0.8 of the vertical force. In ddition to the foregoing factors and conditions, the distribution of vertical loads is ffected by the vehicle's spring oscillation. Figure 9 shows interior vertical normal and horizontal shear stresses caused by application o a circular area of a uniform shear stress in one direction, these stresses are in addition to he stresses caused by the applied normal oads.

The stresses shown in figure 9 were derived rom a table of stresses produced by a uniform vertical stress on a circular area (*8*) by the relationships given in the following statements.

p_v/s = the component of horizontal shear stress in the direction of s, divided by the applied vertical stress.

s_h/s = the horizontal normal stress (for Poisson's ratio = 0.5) in the direction of s, divided by the applied vertical stress.

The differences in the maximum stresses on a horizontal plane at different depths are shown in figure 10. The horizontal shear

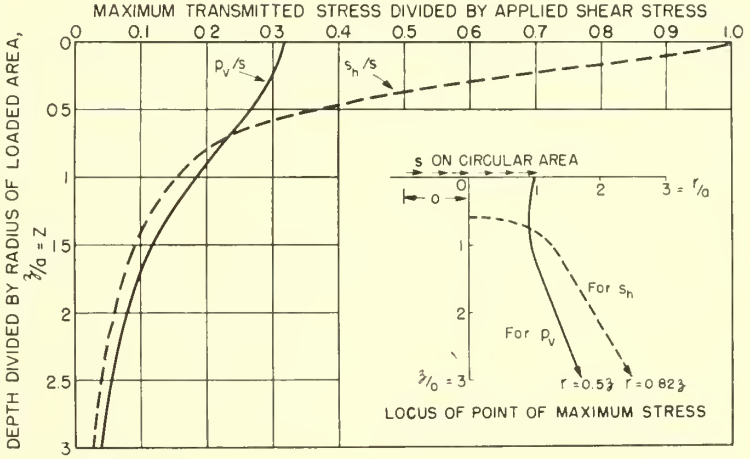
MAXIMUM TRANSMITTED STRESS DIVIDED BY APPLIED SHEAR STRESS
DEPTH DIVIDED BY RADIUS OF LOADED AREA, $z/a = Z$
p_v/s
s_h/s
s ON CIRCULAR AREA
For s_h
For p_v
$r = 0.5\,a$
$r = 0.82\,a$
LOCUS OF POINT OF MAXIMUM STRESS

Figure 10.—Maximum stress at different depths from shear in one direction at surface.

$$q = (1 - \tfrac{1}{16})^2 cN_c + (1 - \tfrac{1}{f})^2 waN_a$$

Where, q = Bearing capacity, p.s.i
i = Ratio of horizontal to vertical load
w = Unit weight, p.c.i.
a = Radius of loaded area, inches
f = Coefficient of internal friction
N_c & N_a = Functions of f, see table below

f	N_c	N_a
0	7.4	0.0
0.1	9	0.1
0.2	14	0.4
0.3	19	1.2
0.4	27	3.0
0.5	36	6.6
0.6	53	13
0.7	75	26
0.8	106	51
0.9	156	102
1.0	224	192

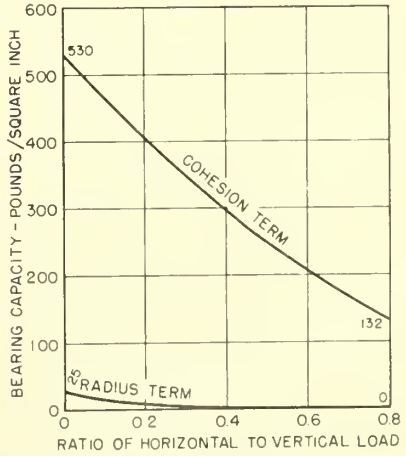

Figure 11.—Effect of horizontal force on bearing capacity.

stress is an important factor that affects the stability of layered systems, and its effect should be considered carefully for construction planned at locations where the bond between layers may be critical.

The information presented in the foregoing paragraphs should be useful when overstress of a few points must be considered. But when the overall failure of a pavement is to be considered, the theory of plasticity should be used to calculate the bearing capacity.

The effect of horizontal load components on bearing capacity developed by Meyerhof (*9*) is shown by the formula in figure 11; this illustration has been included here because the values shown for the coefficient of friction and its function might apply to the bearing capacity of bituminous pavement material. A bearing capacity of 530+25=555 p.s under a vertical load will be reduced 132+0=132 p.s.i. at impending skidding of vehicle. The 132 p.s.i. is the resultant effe of vertical and horizontal applied stress When horizontal and vertical stresses a applied together, their combined effect mu be considered in any analysis of beari capacity of a pavement.

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Relation of Absolute Viscosity of Asphalt Binders to Stability of Asphalt Mixtures

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This article presents the results of laboratory compressive strength tests made to show the relationship of asphalt viscosity to the stability of pavement mixtures. A summary of the more important conclusions drawn by other authors in previously reported research is included in appendix I.

The results reported here illustrate the significant effect of the type and gradation of aggregate on compressive strengths of asphalt mixtures and the variation of strength that can occur at different temperatures for mixtures made with asphalts of the same penetration grade but with different viscosity-temperature susceptibilities. When comparisons were made at the same temperature, mixtures of a particular asphalt and the crushed stone aggregate used in these studies had significantly higher strengths than mixtures of the same asphalt and the gravel aggregate, which in turn had higher strengths than the mixtures made with sand. Such differences in strengths of the crushed stone, gravel, and sand mixtures were indicated to be more significant at temperatures around 140° F. than at lower temperatures.

When comparisons of compressive strengths were made on the basis of equal absolute viscosities, the differences caused by the three types of aggregates were still significant, but the strength of mixtures made with the same type and grading of aggregate and the same percentage of asphalt were equal for equal viscosities regardless of the source and grade of the asphalt.

Introduction

THE MAJOR goal of the asphalt paving technologist is to design asphalt paving mixtures that will have sufficient stability to resist displacement of the asphalt pavement in the form of shoving or rutting under traffic and also have the proper amount of flexibility, skid resistance, and durability. Many research studies have been conducted to evaluate such factors as the type, quality, and gradation of aggregates, and the character and amount of bituminous binder and construction variables that affect the quality and performance of the pavement.

The consistency of the asphalt binder in paving mixtures is one of the most important variables that affects performance. Consequently, the penetration test and other empirical consistency measurements were developed. Since then it has been common practice to specify certain penetration grades of asphalt for paving mixtures for use under specific conditions of traffic and climate. Initially, the grade of asphalt was selected on a trial and error basis; the lower penetration materials were used in warm areas and for heavy traffic conditions, and the higher penetration asphalts were used in cold areas and for lighter traffic conditions.

During the 1930's, the great amount of cracking in pavements precipitated a trend toward the use of softer asphalts (higher penetration) in sheet asphalt and asphalt concrete construction in many areas of the country. Since that time, laboratory stability tests have come into general use for designing paving mixtures. Using these tests, the researcher and engineer have developed a much better knowledge and understanding of the properties of aggregates, mineral fillers, and bituminous binders that influence the stability of various types of paving mixtures. Several research studies that show the effect of asphalt consistency on stability in laboratory tests have been reported.

A review of the literature reveals that these studies generally agree with respect to the qualitative relationships involved. However, the quantitative effects of type and gradation and consistency of the asphalt binder on stability have not been well defined. Consistency, measured by both penetration and viscosity, has been used in the various studies. Some authors have indicated that for a given mixture no difference in stability should occur when asphalts of the same consistency are used. However, a number of others have reported that other properties of the binder, as well as consistency, have a significant effect on stability. No specific information has been found that showed the relative effect of asphalts of different viscosity-temperature or shear-susceptibility characteristics in mixtures made with different aggregates where internal friction differed widely. The study reported in this article was undertaken to obtain such information. The relation shown is the effect of viscosity on the stability of pavement mixtures measured by laboratory test. No attempt was made to correlate the test data with characteristics of the pavement in place, but it is believed that the relative effects indicated will serve as valuable criteria for further studies on the design of mixtures in the laboratory and for predicting the performance of asphalt pavements in service.

This article also includes summaries of some of the more important research reports, which were reviewed to provide the background information for this study. These summaries are given in appendix I and are arranged in approximate chronological order to provide a general picture of the development of knowledge concerning the basic relationships involved.

[1] Presented at the 65th annual meeting of the American Society for Testing and Materials, New York, N.Y., June 1962.

Table 1.—Source, method of refining, and test characteristics of asphalts

Asphalt sample	Source of crude	Method of refining	Penetration, 100 g., 5 sec. at 77° F.	Ductility, 5 cm./min. at 77 F.	Softening point	Furol viscosity at 275° F.	Specific gravity at 77° F.	Thin-film oven test, ⅛-in. film at 325° F., 5 hrs.			
								Loss [1]	Tests on residue		
									Softening point	Ductility, 5 cm./min.	Percent of original penetration
				Cm.	*°F.*	*Sec.*		*Percent*	*°F.*	*Cm.*	*Percent*
3	Mexico	Atmospheric steam distillation.	87	248	121	318	1.037	0.55	134	138	60
13	Venezuela	Unknown	89	175	120	255	1.033	.18	132	175	62
38	Midcontinent	Vacuum distillation to 225–275 penetration, blown at 480° F.	90	170	120	189	1.000	.00	129	120	62
56	Kansas	Vacuum distillation.	86	166	121	180	1.004	+.03	139	13	62
69	Texas	Steam-vacuum distillation.	87	112	116	120	1.029	+.02	125	178	61
100	California	Steam distilation.	88	245	120	228	1.031	1.03	136	125	49
121	Mexico	Steam distilation.	57	>250	128	431	1.039	.29	140	100	65
154	Texas	Steam-vacuum distillation.	63	212	122	144	1.033	+.04	127	176	62
196	Mexico	Steam distilation.	135	184	113	250	1.034	.93	129	235	50
226	Texas	Steam-vacuum distillation.	126	118	109	86	1.023	.01	118	120	57

[1] +sign indicates gain in weight.

Materials Used

The asphalts used for the tests reported here were selected from the group included in the studies of the properties of highway asphalts produced in the United States reported by the Bureau of Public Roads in 1959 and 1960 (*1*, *2*). Table 1 provides information, which was given in earlier reports, on the sources of the asphalt, methods of refining, and some of the more important physical characteristics of the asphalts. The same identification number for each sample has been used in this article to permit convenient cross-reference. Table 2 shows the composition of the selected 85–100 penetration asphalts and also the changes in composition during aging tests conducted by Rostler and White, which they reported at the 1962 meeting of the Association of Asphalt Paving Technologists (*3*). The composition shown was based on the Rostler-Sternberg method in which the asphaltenes are first precipitated with normal pentane and the dissolved fractions are treated successively with sulfuric acid of specified strength. It is believed that the asphalts selected for this study and described in the following paragraphs, had sufficiently different characteristics so that their behavior would provide a general cross section of all types of commercial-grade asphalt materials.

Table 2.—Composition of 85–100 penetration asphalt [1]

Asphalt sample [2]	Composition [3]					$\frac{N+A_1}{P+A_2}$	Durability rating
	A	N	A_1	A_2	P		
3:							
Original	30.7	16.0	20.6	23.7	9.0	1.12	2
Mixed	34.3	16.9	18.1	22.2	8.9	1.14	
Aged	36.0	18.3	16.2	21.5	8.0	1.17	
13:							
Original	28.8	22.5	23.0	18.9	6.8	1.77	5
Mixed	32.0	23.5	18.5	19.1	6.9	1.61	
Aged	36.0	24.8	17.1	15.5	6.6	1.90	
38:							
Original	20.3	18.4	23.8	24.8	12.7	1.13	2
Mixed	24.6	16.2	21.9	26.7	10.6	1.02	
Aged	30.2	16.5	19.7	23.8	9.8	1.08	
56:							
Original	20.3	12.1	21.0	30.8	15.8	0.71	1
Mixed	22.7	12.4	21.3	29.7	13.9	0.77	
Aged	25.0	11.9	14.8	34.7	13.6	0.55	
69:							
Original	21.0	14.6	23.9	31.0	9.5	0.96	7
Mixed	22.7	20.4	23.0	24.4	9.5	1.28	
Aged	25.7	20.7	27.5	26.8	9.5	1.05	
100:							
Original	33.4	21.7	23.1	13.2	8.6	2.06	7
Mixed	55.6	21.7	19.7	16.3	6.7	1.80	
Aged	41.8	20.7	16.5	14.1	6.9	1.87	

[1] Taken from paper by Rostler and White presented at meeting of Association of Asphalt Paving Technologists, New Orleans, La., January 1962.

[2] Asphalt:
Original, as received.
Mixed, after mixing with Ottawa sand 6 minutes at 325° F.
Aged, after aging 7 days at 140° F.

[3] Composition determined by Rostler-Sternberg method:
A = Asphaltenes.
N = Nitrogen base resins.
A_1 = First acidaffins.
A_2 = Second acidaffins.
P = Paraffins.

The series of Texas asphalts, samples 154, 69, and 226 were respectively 60–70, 85–100, and 120–150 penetration grades from the same producer, and they represented the most temperature-susceptible materials found in the previous studies, where three grades were available from the same source. Some 85–100 penetration grade asphalts from California sources have lower furol viscosities at 275° F than sample 69, the 85–100 material in this series, but the difference is not great.

The series of Mexican asphalts, samples 121, 3, and 196, were 60–70, 85–100, and 120–150 penetration grades and represented the materials with the lowest viscosity-temperature susceptibility tested in the earlier study.

The other four asphalts selected for this study were of the 85–100 grade: (1) Sample 13, a Venezuelan material, was typical of much of the asphalt used on the east coast. (2) Sample 38 was a midcontinent material that was highly resistant to hardening in both the microfilm and thin-film tests reported to ASTM in June 1961 (*4*), and this material had a relatively high shear susceptibility. (3) Sample 56 material also had a high shear susceptibility and the ductility of its thin-film residue was very low (*4*). Although Rostler and White (*3*) reported that their test showed this material had excellent abrasion resistance, performance of asphalt having similar characteristics and believed to have been obtained from the same source was very poor. (4) Sample 100 was a California asphalt, which had a relatively high loss and a low percentage of retained penetration in the thin-film test.

The aggregates used in this study were selected to give test mixtures of three distinct types having widely different stability characteristics. These are designated in this article as sand, gravel, and stone mixtures. The aggregate for the sand mixture was a blend of concrete and sheet asphalt sands and limestone dust. The aggregate for the gravel mixture was composed of uncrushed gravel, natural sand, and limestone dust. The aggregate for the stone mixture was entirely crushed stone, including the material passing the number 200 sieve. The compositions of the test mixtures, including the type and gradation of the aggregates are shown in table 3. Although the gradations chosen for each type of aggregate were generally within limits that are used for actual pavement construction, it should not be assumed that these gradations represent the best combinations for each type. The primary basis for selection was to obtain aggregate systems having low, medium, and high levels of internal friction.

Table 3.—Composition and aggregate grading of test mixtures

	Sand mixture	Gravel mixture	Stone mixture
Asphalt, aggregate basis	*Percent* 8.5	*Percent* 6.0	*Percent* 6.0
Aggregate type:			
Red Hill granite			100
Massoponax gravel		58	
Potomac River sand		36	
White Marsh concrete sand	60		
Potomac River sheet asphalt sand	38.5		
Limestone dust	1.5	6	
Aggregate passing sieve:			
1½ inch		100	
⅜ inch		90	100
No. 4	100	62	82
No. 10	90	42	36
No. 20	72	32	
No. 40	52	19	13
No. 80	17	9	9
No. 200	4	6	4

Variation of Compressive Strength with Temperature

Results of the compressive strength test, made at various temperatures on specimens 3 inches in diameter and 3 inches high in which each of the aggregates were used with each asphalt, are shown in table 4. The details of mixing, molding, and testing these specimens are given in appendix II. The compressive strength result reported for each asphalt at each test condition is the average for four individual specimens. The bulk specific gravity and the voids content of the mixtures given in the table also are averages for the group of four specimens tested. Figure 1 shows the effect of temperature on compressive strength of the three types of mixtures containing asphalts 3 and 69—the 85–100 penetration grade asphalts representing the extremes of temperature-susceptibility of the asphalts used in this study.

As expected, a large difference was noted in the compressive strength of the three types of mixtures, especially at the higher temperatures. For example, at 120° F. the sand mixture for asphalt 69 had a strength of 5 p.s.i.; the gravel mixture had a strength of 32 p.s.i. (interpolated); and the crushed stone mixture had a strength of 102 p.s.i. At lower temperatures the differences were less pronounced. The strength at 60° F. for asphalt 69 was: 327 p.s.i. for the sand mixture; 440 p.s.i. (interpolated) for the gravel mixture; and 486 p.s.i. for the stone mixture. Differences in the compressive strength of mixtures containing asphalt 3 were of about the same order of magnitude. The flatter slope of the plotted line for the crushed stone mixtures reflects the greater interlocking or internal friction present in the crushed stone aggregate.

The same type of aggregate mixtures made with asphalts of the same penetration grade but from different sources also had significantly different compressive strengths. The differences in strength caused by differences in asphalt source were greatest with the sand

Table 4.—Compressive strengths of mixtures made with different asphalts and aggregates

Asphalt sample	Type of aggregate [1]	Bulk sp. gr.[2]	Air voids,[2] percent	Compressive strength, p.s.i., at—[2]						
				40° F.	59° F.	60° F.	82° F.	95° F.	100° F.	120° F.	140° F.
154	Gravel	2.255	7.4	1,228	585	----	205	----	90	----	22
69	Sand	1.939	18.2	708	----	327	----	33	----	8	----
	Gravel	2.256	7.3	1,084	469	[3] 440	166	[3] 90	71	[3] 32	16
	Stone	2.365	8.1	1,383	----	486	----	169	----	102	----
226	Gravel	2.251	7.7	774	327	----	130	----	48	----	11
121	Gravel	2.275	6.7	999	547	----	252	----	136	----	50
3	Sand	1.953	17.7	558	----	273	----	55	----	21	----
	Gravel	2.272	6.7	785	396	[3] 390	192	[3] 125	106	[3] 59	38
	Stone	2.380	7.4	959	----	411	----	193	----	133	----
196	Gravel	2.270	6.9	676	320	----	146	----	75	----	23
13	Sand	1.955	17.4	----	----	298	----	48	----	14	----
	Gravel	2.273	6.2	----	----	428	----	122	----	54	----
	Stone	2.383	7.3	----	----	448	----	177	----	123	----
38	Sand	1.944	17.3	----	----	256	----	38	----	12	----
	Gravel	2.263	6.2	----	----	362	----	110	----	46	----
	Stone	2.355	7.6	----	----	382	----	188	----	122	----
56	Sand	1.941	17.2	----	----	249	----	72	----	29	----
	Gravel	2.267	6.4	----	----	366	----	139	----	78	----
	Stone	2.372	7.2	----	----	381	----	198	----	140	----
100	Sand	1.958	17.2	----	----	324	----	56	----	18	----
	Gravel	2.272	6.4	----	----	451	----	132	----	58	----
	Stone	2.383	7.3	----	----	554	----	197	----	130	----

[1] See table 3 for grading and composition of mixtures.
[2] Each value is an average of results for 4 test specimens.
[3] Interpolated strengths.

Table 5.—Viscosities of original and recovered asphalts, 85–100 grade, at various temperatures

Asphalt sample and test temperature	Viscosity [1]			
	Original asphalt	Recovered asphalt from—		
		Sand mixture	Gravel mixture	Stone mixture
	° F.	*Poises*	*Poises*	*Poises*	*Poises*
3:	40	1.2×10^8	----	1.9×10^8	----
	59	8.7×10^6	----	2.0×10^7	----
	60	8.3×10^6	2.0×10^7	----	2.0×10^7
	77	1.1×10^6	3.4×10^6	3.0×10^6	2.8×10^6
	82	5.2×10^5	----	1.8×10^6	----
	95	1.3×10^5	4.7×10^5	----	3.9×10^5
	100	8.5×10^4	----	3.4×10^5	----
	120	1.5×10^4	5.5×10^4	----	3.9×10^4
	140	3.3×10^3	----	1.8×10^4	----
13:	60	1.1×10^7	1.7×10^7	1.5×10^7	1.7×10^7
	77	1.2×10^6	2.4×10^6	2.1×10^6	2.2×10^6
	95	2.3×10^5	3.0×10^5	2.4×10^5	2.7×10^5
	120	1.8×10^5	3.0×10^4	2.4×10^4	2.4×10^4
38:	60	1.0×10^7	1.5×10^7	1.5×10^7	1.3×10^7
	77	1.2×10^6	2.9×10^6	2.1×10^6	1.9×10^6
	95	1.3×10^5	3.3×10^5	2.0×10^5	2.3×10^5
	120	1.3×10^4	4.0×10^4	2.0×10^4	1.6×10^4
56:	60	8.0×10^6	1.3×10^7	1.9×10^7	1.6×10^7
	77	1.7×10^6	3.8×10^6	3.7×10^6	3.7×10^6
	95	2.1×10^5	6.5×10^5	4.6×10^5	6.3×10^5
	120	3.4×10^4	7.0×10^4	6.8×10^4	9.3×10^4
69:	40	5.2×10^3	----	1.2×10^4	----
	59	1.8×10^7	----	3.2×10^5	----
	60	1.6×10^7	3.0×10^7	----	2.6×10^7
	77	1.0×10^6	2.9×10^6	2.1×10^6	2.6×10^6
	82	5.7×10^5	----	1.1×10^6	----
	95	1.1×10^5	2.6×10^5	----	2.7×10^5
	100	6.6×10^4	----	1.0×10^5	----
	120	8.3×10^4	1.8×10^4	----	2.0×10^4
	140	1.5×10^3	----	2.1×10^3	----
100:	60	1.1×10^7	2.4×10^7	2.3×10^7	2.2×10^7
	77	1.2×10^6	3.1×10^6	2.4×10^6	3.3×10^6
	95	1.9×10^5	3.8×10^5	3.5×10^5	3.9×10^5
	120	1.9×10^4	3.3×10^4	3.6×10^4	3.5×10^4

[1] Shear rate 0.05 sec.$^{-1}$

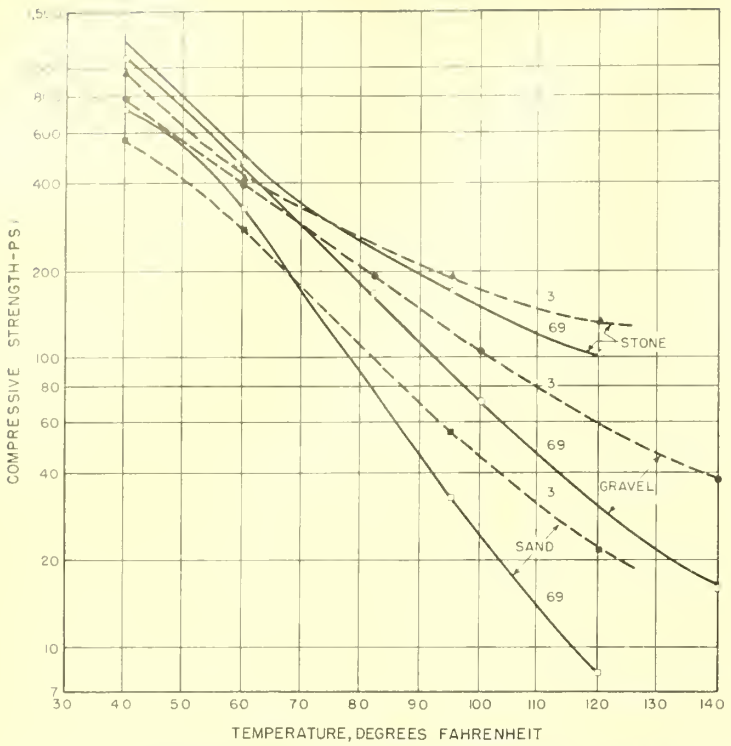

Figure 1.—Relation between log strength and temperature—different aggregates and asphalts.

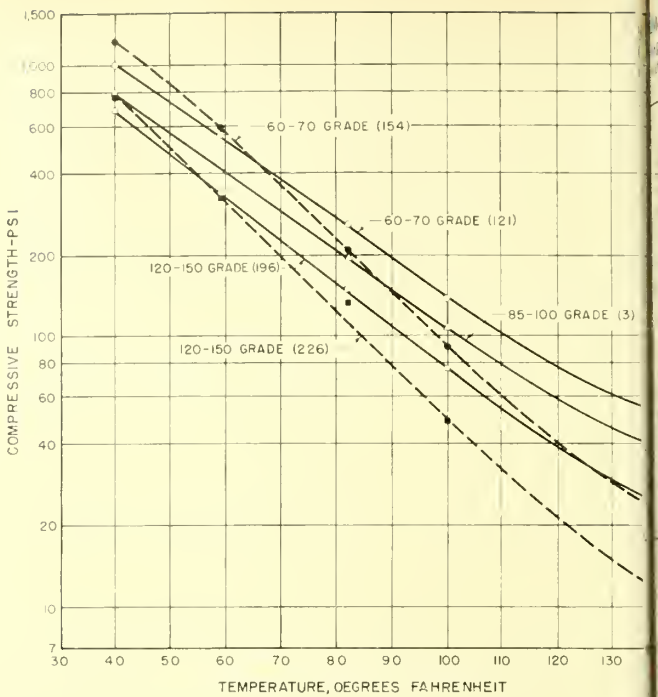

Figure 2.—Relation between log strength and temperature asphalts of different grades.

mixture and least with the stone mixture. For example, the strength of the sand mixture at 120° F. containing asphalt sample 3 was 2.6 times the strength of a similar mixture containing asphalt 69. At this temperature the ratios of the strengths of the gravel and stone mixtures were 1.8 and 1.3 respectively for the same two asphalts.

The variation in compressive strength with temperature for gravel aggregate mixtures made with asphalts from the same source but of different penetration grades is illustrated in figure 2. The data plotted are for the mixtures containing 60–70 and 120–150 penetration grades of the Mexican and Texas asphalts, representing the extremes of temperature-susceptibility. The curve for mixtures made with the 85–100 Mexican asphalt is also included for comparison. As shown, the curves for different grades from a given source are substantially parallel but differ significantly in slope from the curves for mixtures made with corresponding grades of asphalt from a different source.

The relative position of the curves for the mixtures containing 60–70 Texas asphalt sample 154, and those containing the 120–150 Mexican asphalt sample 196, are of interest. These mixtures had approximately the same compressive strengths at 140° F., but at 40° F. the strength of the mixture containing sample 154 was almost double that of sample 196. However, the mixtures containing asphalt samples 3 and 226 had the same strengths at 40° F. but widely different strengths at 140° F. (Sample 3 was the 85–100 Mexican asphalt and sample 226 the 120–150 Texas asphalt.) These comparisons illustrate the effect of the temperature-susceptibility of asphalts on mixture strengths.

The general relations shown in figures 1 and 2 are well known from the qualitative standpoint and will be exhibited by all types of mixtures, but the amount of change in strength will differ and depends on both the asphalt characteristics and the differences in aggregate such as grading, particle shape, etc. The degree of compaction of the laboratory specimen and the percentage of asphalt used also will influence the strength relationships obtained with each mixture composition. Because the effect of asphalt viscosity was the primary consideration in this study, evaluation was made of the quantitative effe of other factors.

Table 6.—Viscosities of original and recovered asphalts, 60–70 and 120–150 grades, at different temperatures

Test temperature, ° F.	Viscosities [1] for 60–70 grade asphalts—		Viscosities [1] for 120–150 grade asphalts—	
	Original	Recovered [2]	Original	Recovered [2]
	Asphalt 121		Asphalt 196	
40	3.4×10^{8}	3.5×10^{8}	6.1×10^{7}	3.3×10^{8}
59	2.9×10^{7}	3.6×10^{7}	4.6×10^{5}	1.1×10^{7}
77	3.1×10^{6}	5.6×10^{6}	5.8×10^{5}	1.3×10^{6}
82	1.3×10^{6}	3.6×10^{6}	2.3×10^{5}	6.9×10^{5}
100	2.0×10^{5}	6.8×10^{5}	4.4×10^{4}	1.1×10^{5}
140	6.3×10^{3}	3.6×10^{4}	1.7×10^{3}	4.2×10^{3}
	Asphalt 154		Asphalt 226	
40	1.2×10^{9}	2.4×10^{9}	1.6×10^{8}	5.8×10^{8}
59	3.9×10^{7}	6.9×10^{7}	6.7×10^{6}	1.4×10^{7}
77	2.3×10^{6}	4.8×10^{6}	3.5×10^{5}	8.8×10^{5}
82	1.1×10^{6}	2.2×10^{6}	2.4×10^{5}	4.6×10^{5}
100	1.3×10^{5}	2.1×10^{5}	3.1×10^{4}	4.2×10^{4}
140	2.5×10^{3}	4.4×10^{3}	7.8×10^{2}	8.5×10^{2}

[1] For shear rate of 0.05 sec.$^{-1}$
[2] Asphalt recovered from gravel mixture.

Variations of Compressive Strengt with Viscosity

To show the extent to which the differenc in stability of bituminous mixtures determin in the laboratory by the compressive streng test can be attributed to differences viscosity of the contained asphalt, the asphal were recovered from test specimens represe ing each type of mixture and their viscositi

ble 7.—Comparison of penetration of asphalts recovered from test specimens with penetration of original asphalts

Asphalt grade and sample number	Penetration, 100 g. 5 sec., 77° F.			
	Original asphalt	Asphalt recovered from—		
		Sand mixture	Gravel mixture	Stone mixture
85–100:				
3	92	61	63	63
13	88	62	66	65
38	89	63	68	70
56	78	54	61	57
69	88	53	61	56
100	90	54	59	58
60–70:				
121	66	-------	47	-------
154	61	-------	43	-------
120–150:				
196	127	-------	91	-------
226	145	-------	89	-------

ere determined at several temperatures. The scosities of the original asphalts also were termined over the same temperature range measure the hardening that had occurred uring the preparation and testing of the ecimens.

All viscosities were determined with the iding plate microviscometer. This instrument has been adequately described in a mber of published reports (*14, 15*). Determinations for each material were made at our temperatures covering the normal working range of the instrument. Although there ere some exceptions, most of the determinations were made at 60° F., 77° F., 95° F., and 10° F. All viscosities were computed on the asis of a shear rate of 0.05 sec.$^{-1}$ These ata were plotted on a viscosity temperature hart using the ASTM coordinates. The asis for this chart is log log viscosity in entistokes, which is essentially equivalent to entipoises for these materials, plotted against og of absolute viscosity in degrees Rankine ° F.+459.7). The viscosities reported in ables 5 and 6 were extrapolated or interolated from the plotted lines based on the est straight line through the determined data oints. Such extrapolated viscosities may ot be exact for the extrapolations at the pper and lower temperatures (120° F. or 40° F. and -40° F.) but they are believed to e sufficiently precise for the purposes of this tudy.

The data in table 5 for the 85–100 penetration asphalts include viscosities of the original sphalts and of the asphalts recovered from nixtures made with each of the aggregates. All of the recovered asphalts had higher iscosities than the original asphalt; and, enerally, the viscosity increase occurring with the use of each type of aggregate was ot significantly different. The greatest difference occurred for asphalt 38. At 120° F. he viscosity of the material recovered from he sand mixture made with this asphalt was pproximately double the viscosity of the material recovered from other aggregate mixtures; but at 60° F. the viscosities for all three recovered asphalts were essentially the same. Table 6 shows the data for both the original and recovered asphalts for the 60–70 and 120–150 grades. Asphalts of these grades were recovered from only the gravel mixtures.

Table 7 contains data that compares the penetrations of the original and recovered asphalts at 77° F. The penetration data provide a more familiar evaluation of the hardening occurring during the mixing and curing of the various specimens and show that the increase in consistency, measured by the penetration at 77° F., is equivalent to approximately a one-grade change in penetration. For the 85–100 grade materials, the original penetration at 77° F. varied from 78 to 92. Asphalt sample 56, which had a penetration of 78, was out of grade. The penetration of the recovered asphalts ranged from a low of 53 for the asphalt recovered from the sand mixture containing asphalt sample 69 to a high of 70 for the asphalt recovered from the stone mixture containing asphalt sample 38. Generally, the hardening during the mixing and curing of the sand mixtures, measured by penetration at 77° F., was greater than that for the gravel and stone mixtures.

In figure 3, the logarithm of the compressive strength at each temperature is plotted against the logarithm of the viscosity of the recovered asphalt at the same temperature. Regardless of the source or penetration grade of the asphalt used, the data points fall on three distinct curves, one for each type of aggregate mixture. The influence of the aggregate system in the mixtures is again illustrated by the different levels and slopes of the curves. Although there is some curvature in the plotted lines, small segments can be considered to be straight and the slopes of such segments can be used to indicate the ratio of change in strength to the change in viscosity. Examination of these slopes in the range of viscosity from 10^4 to 10^5 poises, which is approximately equivalent to viscosities at summer temperatures, shows that a 10-fold increase in viscosity produced approximately a 3.2-fold increase in strength for the sand mixture, a 2.4-fold increase for the gravel mixture, and a 1.5-fold increase for the stone mixture. Corresponding strengths for viscosities such as would exist for winter temperatures in the range of from 10^8 to 10^9 poises could not be estimated precisely. However, it is probable that further increase in viscosity produced by decrease in temperature would not cause as large an increase in strength. This tendency is indicated by the curvature of the plotted line for the sand mixture and, to a lesser extent, by the curvature of the plotted line for the gravel mixture. The data for the stone mixture was less definite, but it is believed that strengths at higher viscosities than obtained in this study would show similar behavior of the mixture.

Other researchers have studied the relation of asphalt viscosity to laboratory determined stability and some have indicated that a straight line is the result when log strength is plotted against log log viscosity. Summaries of three studies using this relation are included in appendix I; these studies were by Weetman and Hurlburt (*7*), Neppe (*8*), and Wood and Goetz (*9*). To examine this relationship, the data obtained in this study (tables 4, 5, and 6) were plotted as shown in figure 4. It should be noted that, although the horizontal axis of the plot shows viscosity in poises, the basis for locating the points was the log log viscosity in centipoises. The data for the gravel mixtures with all asphalts fall on a straight line over the entire viscosity range explored. The plotted data for the sand and stone mixtures made with all asphalts closely approximate straight lines for viscosities less than 10^7 poises, but for greater viscosities the lines show some curvature.

Some authors have indicated that factors other than viscosity have a significant effect on strength measured by laboratory stability tests. It is obvious from figures 3 and 4 that much of such an effect, if it existed, would be masked by aggregate systems having good inherent stability. Special attention therefore was directed to the results obtained with certain asphalts in the sand and gravel mixtures. Figure 5 shows viscosity and strength data for three asphalts replotted on the same basis used in figure 4. These data are for asphalts 3 and 69, which represented the extremes of temperature-susceptibility, and asphalt 56, which had the greatest degree of complex flow. Although tests were made at only three temperatures for the sand mixture prepared with each asphalt, there were some indications that the data points would form a slightly different line for each asphalt. Data were available at five temperatures for the gravel-mixtures for asphalt samples 3 and 69 and, as indicated, no difference in the plotted line could be detected. There was some indication that the three data points for the gravel-mixture curve for asphalt sample 56 formed a slightly different line than was obtained from data points for asphalt samples 3 and 69, but this difference might have been the result of experimental error.

When evaluating the significance of this difference, the possible effect of the shear rate used to determine the viscosity of the asphalts compared to the shear rate of the asphaltic film during the strength test should be considered. As previously stated, all of the viscosities reported in this study were based on a shear rate of 0.05 sec.$^{-1}$ This basis was selected because this shear rate is the one most commonly used with the sliding plate microviscometer. Also, this shear rate could be determined with relatively good precision for the tests made on these asphalts as it could be bracketed by actual test results. This rate of shear, however, did not correspond to the rate of shear in the mixture specimen during the strength test. Based on the average film thickness of the asphalt in the mixture and the speed of the test, the rate of shear in the compressive strength specimen at failure most likely was in the range of 1 sec.$^{-1}$ to 2 sec.$^{-1}$

Attempts were made to extrapolate the viscosity test data so that the apparent viscosity could be calculated on the basis of

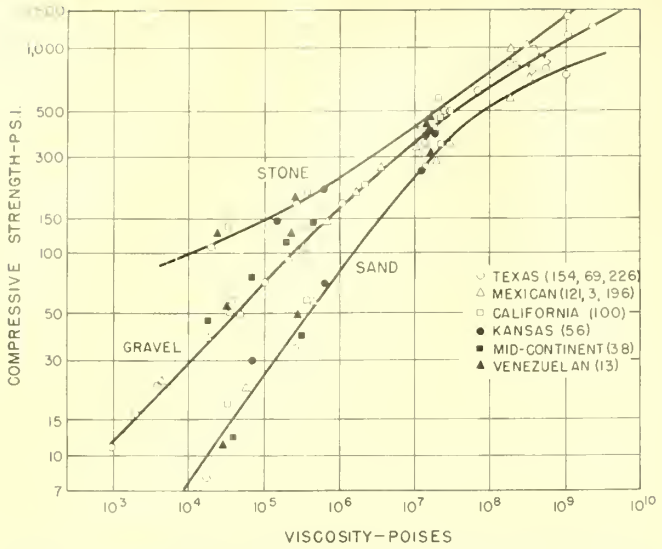

Figure 3.—Relation between log strength and log viscosity of asphalt.

the estimated shear rate existing during the test, but the extent of the extrapolation so greatly reduced the precision of the estimation that the results proved to be of no value. It is entirely possible that more precise measurements of strength would show greater differences in strength for the same viscosity than were indicated in this study, but it is believed that more study is needed to determine whether such differences are significant with respect to pavement behavior.

Discussion of Results

Essentially all of the relationships shown by the tests conducted in this study have been indicated in previous research reports; some of these reports are summarized in appendix I. However, such previous studies were limited in scope as to tests over a wide range of temperatures on mixtures made with different distinctive types of aggregate systems and asphalts from different sources. The data reported in this article are believed to be of considerable value as they provide a quantitative estimate of the effect of the preceding factors. The variations in strengths of mixtures produced by differences in viscosity-temperature susceptibility of the asphaltic binder were much greater for the sand mixtures than for the gravel or crushed stone mixtures selected for this study. These differences were most pronounced at higher temperatures; but, as the temperatures decreased the differences between aggregates became less important.

When comparisons were made on the basis of the same viscosity, regardless of source or penetration grade, the relative differences in the strengths of the sand, gravel, and stone mixtures were greatest at the maximum service temperature (140° F.) at which the viscosities were lowest. The relative difference decreased with a decrease in temperature or an increase in viscosity. For viscosities of about 10^9 poises or higher, the effects of aggregate or possible differences in asphalts were not clearly evaluated in this study. There were some indications that the curve for the sand mixture leveled off; that is, it approached a maximum strength, but the curves for the gravel and stone mixtures did not show this tendency. It is believed that at such high viscosities, asphalts may cease to behave as viscous liquids and act more nearly as solids. Thus, tests to measure properties such as brittleness, stiffness, or fatigue resistance may provide a better evaluation of mixture properties at temperatures near 40° F. or lower than compressive strength tests such as were used in this study.

In certain aspects, the data from this study support the suggestion made by some asphalt technologists that asphalt cements should be graded on the basis of their viscosities at 140° F. rather than on penetrations at 77° F. Differences in viscosities of asphalts at 140° F. have been shown to be much more critical insofar as compressive strength is concerned than viscosity differences at lower temperatures.

Tests in this study showed no significant differences in compressive strengths for mixtures made with asphalts of the same viscosity from different sources. There was some indication that different asphalts having the same viscosity may produce slightly different strengths for weak aggregate systems at high temperatures, but differences of the order indicated would have no practical significance. Even these small differences were minimized at low temperatures.

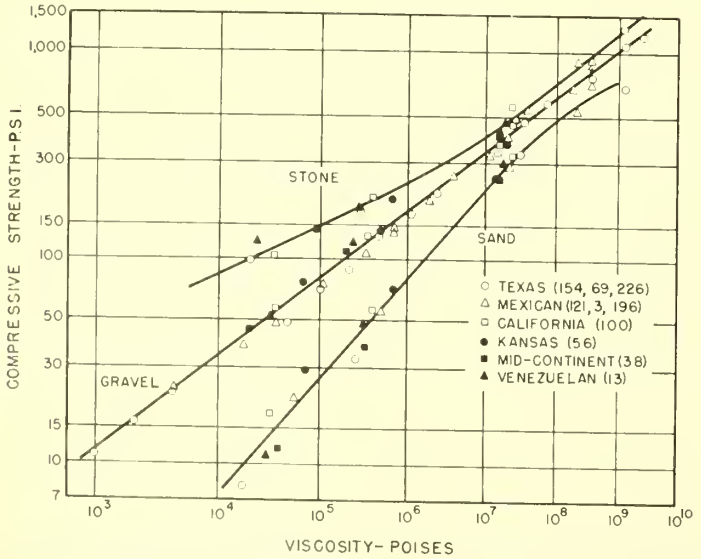

Figure 4.—Relation between log strength and log log viscosity of asphalt.

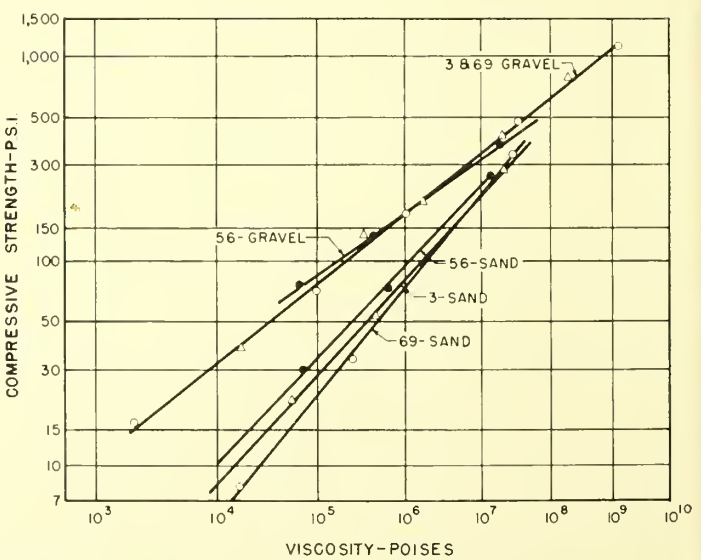

Figure 5.—Relation between log strength and log log viscosity of selected asphalts.

Summary of Previous Research

To provide background information for the work reported in this article, a literature survey was made. The following listed papers have been summarized to illustrate the general development of information and the present state of knowledge concerning the various relationships of viscosities and stabilities of asphaltic mixtures. Only a brief summary and major conclusions from each paper have been included. For a more detailed description of the research, the complete article should be referred to.

Stability Experiments on Asphaltic Paving Mixtures

In 1934, Emmons (*10*) reported a study conducted by the Bureau of Public Roads to establish performance data for the development of laboratory strength tests for bituminous pavement mixtures. Experimental pavement sections were constructed of many different compositions on a circular roadway 180 feet in mean diameter. The performance of these sections was observed under controlled-traffic conditions. Asphalts having different penetrations were used as binders in sheet asphalt and asphaltic concrete paving mixtures. Observations on the performance of the test sections showed that stable mixtures for sheet asphalt had been obtained from the use of asphalts having penetrations of 35 to 55. Use of softer, steam-refined asphalts having penetrations of 63 and 72 caused somewhat more plastic mixtures; however, only one section of test pavement had been laid that contained each of these asphalts. The consistency of the asphalt cement had no apparent influence on the service characteristic of the coarse-graded asphaltic concrete. Stable mixtures were laid with asphalts having penetrations ranging from 45 to 75.

Effect of Consistency and Type of Asphalt on Hubbard-Field Stability of Sheet Asphalt Mixtures

In 1940, Hillman (*11*) reported the effect of consistency and type of asphalt on the Hubbard-Field stability of sheet asphalt mixtures. Mixtures were made with 10 different grades of Mexican asphalt having penetrations that ranged from 23 to 182 and that were obtained from the same producer, and with two grades of asphalt, 50–60 and 85–100, that were obtained from each of five different sources. The mixtures prepared with the different asphalts were tested for Hubbard-Field stability at temperatures of 104°, 122°, and 140° F.

The findings of this study are summarized, as follows: (1) For asphalts from the same source, the stability of sheet asphalt mixtures depends on the consistency of the asphalt at the temperature of the stability test. (2) There are greater differences in the stability of mixtures made with the same penetration grade of asphalts from different sources than in the stability of mixtures made with 50 and 100 penetration asphalts from the same source. (3) There are some characteristics of asphalt that caused mixtures made with asphalts from different sources to vary in stability even when the consistency of the asphalts, measured by penetration, was the same.

Effect of Characteristics of Asphalts on Physical Properties of Bituminous Mixtures

In 1948, Lewis and Welborn (*12*) reported on the effects of the characteristics of asphalts on the physical properties of bituminous mixtures. This article included results of laboratory tests conducted on a particular bituminous mixture and a review of other investigations. It was concluded that: (1) The strength of a given mixture is dependent upon the consistency of the contained asphalt at the test temperature. (2) The test data from other investigations indicate that there is some characteristic of asphalts that causes comparable mixtures prepared with different asphalts to vary in strength even when contained asphalt has the same consistency.

Flow Properties of Bituminous Binders

In a paper published in 1950, Lee and Warren (*13*) commented on the fact that the flow properties of asphalt or other types of bituminous binders are recognized as being of primary importance in road construction. These authors stated that most of the specification tests, such as penetration, softening point, ductility, and furol viscosity, had no relation to the fundamental nature of flow and that up to that time little reliable information had been established to correlate road behavior with fundamental characteristics of binders.

To develop such fundamental information the Road Research Laboratory (England) obtained viscosity-temperature relations over a temperature range of about 15° C. (59° F.) to 180° C. (355° F.) on a number of asphalts of approximately 65 penetration, which had been obtained from several sources. Rotating-cylinder, Ostwald, and falling-cylinder viscometers were used to cover the temperature range. On the basis of the results of their tests on specimens of road surfacing mixtures made with the different asphalts, the authors concluded that the temperature susceptibility of the road mixture is determined by that of the binder. However, their tests also indicated that differences in the plastic and elastic properties of the asphalts were of more significance in determining the mechanical properties of mixtures than differences in viscosity-temperature coefficients.

Effect of Asphalt Viscosity on Stability of Asphalt Paving Mixtures

Weetman and Hurlburt (*7*), in 1947, investigated the effect of temperature and asphalt viscosity on the punching shear stability of a sheet asphalt mixture. Three asphalts of different penetration grades, from each of two sources, that differed widely in temperature susceptibility were used. Stability of a sheet asphalt mixture in which each asphalt had been used was determined over a temperature range of 40° F. to 160° F. Absolute viscosity of the asphalts recovered from the stability specimens was determined at several temperatures within this range by a coaxial-cylinder viscometer. The more important conclusions drawn from this study are:

- The punching shear stability of sheet asphalt mixtures at temperatures in the range of 40° F. to 160° F. are linear semilog functions of temperature.
- For asphalts of the same penetration at 77° F., the high viscosity-temperature susceptible asphalts had higher stabilities at lower temperatures and lower stabilities at high temperatures than asphalts of lower viscosity-temperature susceptibility.
- Comparable stabilities at 140° F. can be obtained by using a low-susceptible asphalt having an appreciably higher penetration at 77° F. than a highly susceptible asphalt. For the asphalt from the two sources reported, this difference in penetration was approximately 50.
- Stabilities of mixtures made with asphalts from the same source but having widely different penetrations at 77° F. are directly related to the asphalt viscosity at the shearing temperatures. A linear relation between log log viscosity and log stability was found, but the curves for the series of asphalts were not coincident. This indicated that some factor other than viscosity, such as absorption, affected the stability of the sheet asphalt mixture.

Viscosity Effects on the Marshall Stability Test

In 1951, Fink and Lettier (*14*) reported on a study concerned primarily with the effects of the absolute viscosity of asphalt on Marshall stability. Nine asphalts obtained from several sources and of widely different methods of manufacture and temperature-susceptibility were selected for this study.

Viscosities of the asphalts were determined, using the vacuum capillary viscometer, in the range of 120°–205° F. and the Saybolt furol viscometer at higher temperatures. The furol viscosities were converted to absolute viscosities. Marshall stability specimens were made by using one aggregate composition and each of the asphalts. Specimens for mixtures containing each asphalt

were molded at temperatures at which the asphalt had a viscosity of 6 poises, and the specimens were tested at temperatures at which the viscosity was 1,100 poises. This study emphasized the fact that the stability was strongly influenced by the viscous resistance of the binder.

Influence of Rheological Characteristics of Binder on Mechanical Properties of Bitumen-Aggregate Mixtures

In 1953, Neppe (*8*) reported the results of compressive strength tests on mixtures made with aggregate of one type and grading and with 40 asphalt cements having different penetration grades and rheological properties. The author calculated stabilities from the times required for the specimens of the mixture to deform by predetermined amounts during the application of a compressive load that was being increased at a uniform rate. The tests were made at 77°, 104°, 122°, and 140° F. for each asphalt mixture. The viscosities of the asphalts were determined at several temperatures.

The author's primary purpose in this article was to show the usefulness of his "Softening Point Number" as a characterization factor for determining the influence of rheological characteristics of the binder on mechanical properties of mixtures. However, he also reported the effect of the viscosity of the asphalt on compressive strengths of the mixtures. A summary of the study included the following information:

- The results for compressive strengths at 77° F. showed no marked differentiation between mixtures prepared either from bitumens of low or high Softening Point Number.
- The mechanical stability of a bituminous mixture at any temperature is approximately a direct function of the log log viscosity of the contained binder at that temperature and is independent of the source, nature, and proportion of the latter constituent.
- The fluidity characterization factor, which is the numerical difference of log log viscosities at 77° F. and 275° F. multiplied by 100 was shown to be directly proportional to the Softening Point Number. The authors concluded that the Softening Point Numbers must also give a direct measure of the temperature-susceptibility of bitumens.

Relationship Between Unconfined Compressive Strength of a Bituminous Mixture and Viscosity of the Binder

In 1958 Wood and Goetz (*9*) used a compressive strength test to study the effect of asphalt viscosity on mixture stability. One of the two objectives of this research was to determine the relation between the viscosity of the binder and compressive strength of a mixture at several temperatures and rates of shear. The parallel-plate microviscometer was used to determine the viscosities at 40° F., 100° F., and 140° F. over a range of shear rates for four asphalts from different sources. Sheet asphalt mixtures composed of 74 percent sand, 17 percent filler (material passing the No. 200 sieve) and 9 percent asphalt were prepared with each of the four asphalts. Specimens 2 inches in diameter and 4 inches in height were molded by the double-plunger method. Compressive strengt tests were made at 40° F., 100° F., and 140° I at three rates of deformations. Relationshi between viscosity and maximum compressiv strength at different shear rates were deter mined. Conclusions drawn from the stud were:

- The shear rate has a very marked effec on viscosity of some asphalts and little effec on others. The effect of shear rate on viscosit also varies with temperature.
- A plot of log compressive strength versu temperature gives a simple means of evaluat ing the temperature-susceptibility of mixtures
- No direct relationship was found betwee compressive strength of sheet asphalt mixture and the viscosity of the contained asphalt ever though the effects of shear rate were taker into consideration. This indicated that fac tors other than viscosity of the binder have an effect on mixture strength.

Effects of Viscosity in Bituminous Construction

In a paper presented at the 1961 annua meeting of ASTM, Verdi Adam (*15*) showed the effect of asphalt viscosity on the Marshall stability of an asphalt concrete mixture from tests in which several different asphalts had been used. The four asphalt cements used in his tests varied from 60–70 to 120–150 penetration at 77° F. The furol viscosity of these asphalts at 140° F. varied from 3.5×10^4 to 3.6×10^5 (determined by conversion from absolute viscosity at 140° F.). The author showed that a good relationship existed between furol viscosity and Marshall stability, both at 140° F., for the different asphalts.

APPENDIX II

Compression Test Procedure

Mixture Preparation

The aggregates for the gravel and stone mixtures for the compression test were separated on various coarse and fine sieve sizes, including the 200 mesh. Portions of each size were then recombined to meet predetermined gradations in batches of sufficient amounts to mold two specimens. Similar test batches for the sand mixture were prepared by combining concrete sand, sheet asphalt sand, and limestone dust in definite proportions. All batches of aggregates were heated overnight in a forced draft oven at 325° F. The asphalt for each batch was prepared by putting a slight excess of the amount required per batch in a sealed, 8-ounce tin that then was placed in a 300° F. oven 45 minutes prior to the asphalt's being blended with the heated aggregate. The required amount of asphalt was added to the aggregate in a modified Hobart mechanical mixer and the materials were mixed for 2 minutes. Each batch contained slightly more mixture than that required for two, 3-inch by 3-inch cylindrical specimens.

Molding Procedure

After being mixed the amount of material required to mold one specimen was transferred to a heated mold. The bowl with the remaining mixture was placed in the 325° F. oven for approximately three minutes, after which the second specimen was molded. For each specimen, the mixture in the heated mold was spaded once around the inside of the mold, then subjected to a 3,000 p.s.i. compaction load that was held for two minutes by use of the double-plunger technique.

Test Procedure

After removal from the molds the specimens were allowed to cool to room temperature and then were placed in a forced-draft oven at 140° F. for 24 hours. The specimens were then cooled overnight at room temperature and the bulk specific gravity of each was determined by use of the water displacement method. The air voids were calculated on the basis of effective specific gravity of the aggregate determined by the Rice vacuum saturation method. The specimens were placed in air baths controlled at the various test temperatures indicated in table 4. In general, the time in the air bath was one-half hour more than that required for a dummy specimen to reach the desired test temperature, which was measured with a thermocouple inserted in the center of the specimen. The bearing blocks of the testing machine were also placed in the air baths to bring them to the test temperature. The compressive strengths of the specimens were determined at a loading rate of 0.15 inches per minute. Each of the compressive strengths and specific gravities given in table 4 is the average result for four duplicate test specimens.

(*References for this article are listed on page 144.*)

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Long-Range Research and Development Program for Individual Transportation Systems

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

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Public Roads officials believe that the Nation's increasing standard of living and its rapidly changing technology require that a research program be undertaken to develop individual transportation systems to fulfill future requirements. To meet this need, Public Roads has proposed the long-range research and development program for individual transportation systems described in this article. The need for the program and its objectives are spelled out; and the three phases envisioned are outlined: Systems analysis, research and development, and prototype testing.

The systems analysis phase will determine the basic criteria governing the performance of any system of individual transportation and will develop a general systems concept. Because the systems analysis phase will provide the framework for the other two phases of the program, a detailed discussion of this important first phase is presented.

Ultimate objective for the total program is the determination of the most promising integrated systems concepts as a basis for completion of the research from which a prototype or prototypes of individual transportation systems can be developed for evaluation.

The long-range research and development program for individual transportation systems will not be accomplished in a short period of time or by any one agency. Public Roads has formulated the general plan and proposes to undertake the initial phases of the program. As the program develops, it is anticipated that there will be participation by the States, industry, and other interested groups.

Introduction

OUR PRESENT highway transportation system is highly effective for individual transportation. It serves the needs and desires of individuals very well. But complacency is dangerous in view of our rapidly changing technology and ever-increasing standard of living. The Interstate Highway System, when completed in 1972, is expected to alleviate congestion, to decrease travel time between origin and destination, and to contribute to an increase in safety, comfort, and convenience for travelers. But it is important to look beyond the completion of the Interstate System. This Nation must keep ahead of the continually changing demands by improving its transportation systems to satisfy individuals needs in the future.

It should be remembered that the Interstate System, as now being constructed, is the culmination of research and development that was started more than a generation ago. To meet the needs of the future, it is necessary to intensify our research and development efforts by utilizing new technology in a coordinated and integrated fashion. Hence, the Bureau of Public Roads is proposing the long-range research and development program for individual transportation systems described in this article.

A recent statement by Robert F. Baker, Director of the Office of Research and Development, Bureau of Public Roads, summarized this long-range program well:

"The accelerating requirements of the Nation make clear that a systematic, energetic research and development program is essential if the optimum transportation system to meet these needs is to become a reality. This program will define a range of alternative transportation system concepts that offer substantial improvements over present concepts. Initially, the program will consist of an intensive systems analysis to develop the basic criteria governing the performance of any system of individual transportation. The ultimate objective of the program will be to determine the optimum integrated systems concepts and to perform the research needed to develop prototypes for field evaluation."

To initiate the first phase of the program, the systems analysis, a set of specifications has been prepared by the Bureau of Public Roads after consideration of the many alternatives suggested by industry, university, and other transportation specialists. The long-range research and development program for individual transportation systems will not be accomplished in a short period of time or by any one agency. Public Roads has formulated the general plan and proposes to undertake the initial phases of the program. As the program develops, it will broaden to include participation by the States, industry, and other interested groups.

Need for the Program

The program described in this discussion has been evolved from an examination of individual transportation; that is, systems designed for individuals to move themselves or their possessions under their own control. This examination was especially related to the ways in which individual transportation may significantly change to meet the needs and requirements of a society that is itself undergoing rapid change. The program was developed because of the recognition that no transportation system can be permitted to drift, with the hope that it will be adequate indefinitely. No society so dependent on personal mobility as ours can afford such luxury. Hence, this program is concerned with the long-range future of individual transportation.

It is obvious from any examination of the highway transportation system that the purposes for which it exists do not depend upon the peculiar physical characteristics of that system. Highway transportation arose out of random invention and has developed as a system in large measure by trial and error. The ultimate reason for the dominance of the highway transportation system over other transportation systems lies in the fact that it better meets the needs of people for movement today. As shown in figure 1, highway transportation offers the individual the freedom to: (1) adapt his travel to a set of time criteria determined by himself, (2) expand the area that

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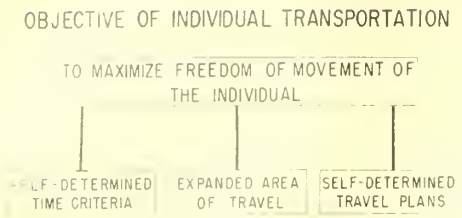
OBJECTIVE OF INDIVIDUAL TRANSPORTATION
TO MAXIMIZE FREEDOM OF MOVEMENT OF THE INDIVIDUAL
SELF-DETERMINED TIME CRITERIA
EXPANDED AREA OF TRAVEL
SELF-DETERMINED TRAVEL PLANS

Figure 1.—Objective of individual transportation.

he can use to satisfy his particular needs, and (3) schedule travel according to his own plan and order of priority. Therefore, regardless of the mechanical methods employed, the objective of any system of individual transportation is to provide maximum freedom of movement so that the greatest possible number of people may satisfy their independent and individual needs for travel and for movement of goods.

Highway Transportation Only One Concept

It should be recognized that highway transportation is only one possible system concept of a tremendous variety of possible concepts that can be employed for individual transportation. Figure 2 shows it to be only one system of a surface-space transportation concept. An air-space concept, of which the ground-effects systems are an example, also can be conceived. Also possible is a time-space concept, of which closed-circuit television is an example. In addition, there may be other concepts that have not been considered, as well as systems formed of combinations of all. Consideration of these concepts poses questions as to whether (1) systems embodying them are technologically possible; (2) how the alternatives are to be defined; and (3) how determinations can be made as to the feasibility of these concepts, and whether the resultant transportation systems would offer measurable improvement over the highway transportation system now available. Many answers to these questions have been and are being suggested. Most, although not all, suggest modifications of the present highway transportation system. Some of the other answers include suggestions for pallet systems or ground-effect systems.

Highway transportation, which may be considered as a system because it operates as the result of the interaction of the three elements of driver, vehicle, and highway, has stimulated suggestions interesting because of their emphasis on one aspect. Almost all suggested modifications have pertained to the driver or, stated more generally, the control mechanism. Suggestions have ranged from driverless automobile systems to complex communication systems.

Although improvement of the existing system by use of sophisticated electronics or mechanical means in the control subsystem is necessary, it is frankly not known whether simply superimposing various devices on highway transportation can ever meet the long-term needs for individual movement. For example, is the control of the vehicle now so poor that new control systems must be added? If so, what kinds of systems? Shall there be a large central computer, which controls groups of vehicles by telemetry, or a small one located in the vehicle? Technologically, use of any of these techniques is possible, but which is the most efficient technique and how can efficiency be defined? Which technique is most reliable and how is its reliability to be measured? Which technique is the safest and how can its safety be proved?

Further, can an optimum solution to the problem be obtained without consideration of the design of other aspects of the highway transport system? What of the vehicle? Can the existing vehicle be modified or can a novel one be substituted that could be controlled more easily or more economically? Can the highway be designed to eliminate control problems? Each of these separate questions may be answered in one way or another. However, it is becoming increasingly obvious that over the long run, a significantly improved system cannot be obtained by treating its parts separately. To achieve a radically improved system of individual transportation, a complete and integrated system must be conceived, designed, and developed. This cannot be done by arbitrarily pursuing any one particular electronic or mechanical technique. Although this approach has been the historical precedent, such a procedure precludes valid comparisons and objective choices among the many possible alternatives.

Limitations of Arbitrary Approach

The limitations of pursuing one electronic or mechanical technique become very evident from a brief analysis of some proposed solutions to the control problem. For example, as indicated in figure 3, induction radio has been developed and is being suggested as a means for transmitting control information to the driver or his equivalent. Another suggested solution involves a system of detector units placed in the roadway that would form electronic blocks for the location of vehicles. However, it should be obvious that to use either of these devices in this manner would imply that a whole set of decisions had been made about the nature of the control problem and its solution. Thus, the use of induction radio techniques would imply a decision to use a system of radio frequency for information transmission rather than some kind of pavement-coding system. It would also imply that a decision has been made about telemetry and radar. In addition, a decision to use induction radio rather than a specialized central computer system would indicate a conclusion that in-auto computers are the way to solve the control problem. Finally, all of these decisions clearly would assume that electronic methods should be used to resolve the control problem.

However, within current limits of understanding of the true nature of the control problem, can mechanical methods of solution

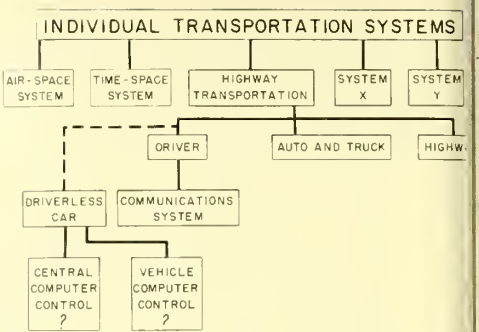

Figure 2.—Highway transportation—on system for individual transportation.

be ruled out? Further, can the current solu tion to system control—the human—be rule out? After all, the human has capabiliti that are difficult to rival mechanically. Fo example, the human can detect angul velocities as low as 5×10^{-5} radians/sec he can discriminate differences in fr quency to an accuracy of 0.2 percent; h can estimate position relative to himself wit an accuracy of 1 percent. These capabiliti not only are unusually good but cost nothin to produce.

This discussion of just one aspect of th highway transportation system shows th tremendous complexity of the problem and th dangers that may arise from the arbitrar choice of one type of solution. This dang obviously applies to all the other aspects the system. To operate in this arbitrar fashion would minimize the chances of ev knowing whether an efficient system had bee selected.

The problem of individual transportatio can be resolved only through a systemat analysis that starts from the essential require ments that any system must have to meet th objectives of individual transportation. Onl from such an objective analysis can measur be developed to rationally evaluate alterna tive physical means so that the most effectiv systems may be selected. To achieve th selection a comprehensive and integrat program of research and development required. Such an approach, which is th modern systems engineering approach, is th one that the Bureau of Public Roads propos to use in the solution of the long-ran problems in individual transportation.

The Program

The proposed program consists of thr phases as shown in figure 4. The first phas is a SYSTEMS ANALYSIS, which will provide framework for the next phase—an intensiv RESEARCH and DEVELOPMENT effort—aimed producing in the third phase one or mo PROTOTYPES for testing. The initial phase the program, the systems analysis, is a pr cedure for defining a complex problem i operational terms. In this way, the proble may be stated in analytical terms, thereb permitting the precise definition of alternativ

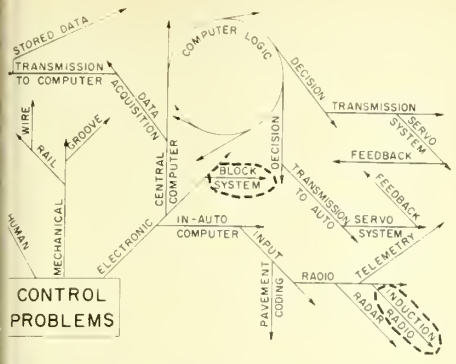

Figure 3.—Possible solutions to control problems.

stems, which can be designed and evaluated. us, the systems analysis will form the mework for the research and development ase.

The research and development phase of the ogram will encompass investigations of the rious components of each of the alternative stems, particularly their interaction. A ntinuing process of evaluation will be used determine whether the alternative systems ected meet the required performance cri- ia. Among other matters, economic con- erations and questions of reliability and blic acceptability of the systems will be vestigated. After evaluation and research ve been done, an intensive development ort is expected to make it possible to pro- le one or more prototype systems for testing. Research, development, and evaluation ll be a continuous and simultaneous process d considerable interaction is expected among ese activities. From these feedback proc- ses, it becomes apparent that the research d development phase will be modified as the stems analysis proceeds. Likewise, the stems analysis will provide a general but xible framework for the research and de- lopment phase.

The third phase of the overall program will nsist of testing one or more prototypes that ve been produced during the research and velopment phase. This testing will be ndertaken on a proving ground before the ototype is subjected to field tests. Again, ere will be feedback between proving ground d field tests.

Similarly, the three phases of the overall ogram are interdependent; and, as men- oned earlier, the research and development ase will undoubtedly be modified from that hich is initially selected. Thus, the systems alysis will, in effect, be modified as research d development proceeds. Similarly, modi- cation also will occur from research and velopment to prototype testing. It is, herefore, conceivable that feedback from ototype testing to the systems analysis could vise the initial systems concept.

THE SYSTEMS ANALYSIS

The first phase in fulfilling the objective this long-range program is to conduct an tensive systems analysis Because the stems analysis is so important in providing direction for the overall program, the remainder of this article will be devoted to it.

Systems analysis is described as the definition of a problem in operational terms, which then permits formulation of a systems concept. The problem is individual transportation, and the goal is to define this transportation system, formulate requirements for it, evaluate and select the most promising systems concepts, and plan for the subsequent phases.

This systems analysis will be essentially a theoretical, analytical effort by a team of engineers and systems analysts. It will not involve physical hardware or its application. It will involve the general or abstract principles of individual transportation. It will formulate mathematical models that present a clear, systematic picture of individual transportation.

This will be the first time that such a comprehensive systems analysis of a transportation system has been undertaken. Its output will provide a better understanding of the overall problem, a logical grasp of the most promising concepts, and identification of critical areas of needed research.

Figure 5 shows that the analysis will consist of three parts—a definition of performance requirements that the system must meet, the formulation of a generalized system concept, and a description of the alternative systems that may be derived from the generalized concept. The first two steps will constitute a purely theoretical study. The general model or concept of individual transportation to be formulated will be the most important, single product of this effort. The alternative systems shown at the bottom of figure 5 will then follow.

At this point it should be added that the relevant user categories that one thinks of today will be considered in the systems analysis. These categories are the transportation of individuals; the mass transportation of people, covering also the movement from origin to the mass transport vehicle and from such a vehicle to a destination; and the transportation of freight together with the special characteristics that such transportation requires.

Performance Requirements

As the first step—definition of performance requirements—a preliminary set of system requirements must be drafted in the early stages of this analysis. Such requirements will be the "guide posts" for the basic evaluation of proposed systems. It should be understood that they are preliminary, however, for they will be continually modified as the program progresses. Experience has shown that formulation of these requirements is a process of achieving a harmonious balance between practical means and ideal goals. A sound requirements statement is therefore an end product of the systems analysis even though in preliminary form it is used for guidance of the study itself.

The statement of performance requirements will define performance criteria that are the measures by which individual transportation can be judged. It will also define the variables or quantities of such a system and their range of values. Some examples of criteria might be the probability of collision, the predictability of position, or the travel time between origin and destination. There may be many others similar to these. There may be other ways of specifying them. However, the criteria must define the system on a complete and rational basis. The variables, and their operating ranges may include such items as speed, flow rate, and size of vehicle. Again these are only examples of the qualities that describe the operation of a generalized concept.

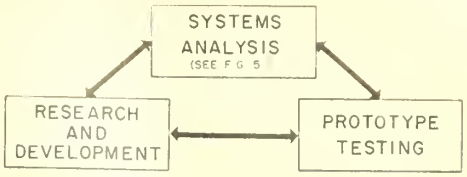

Figure 4.—The three interrelated phases in the long-range research and development program for individual transportation systems.

Looking to the formulation of the generalized system concept, it should be noted that this concept is still completely theoretical and will be based on the performance requirements to be developed.

First, an examination will be made of the essential elements or components of any transportation system. These components include the vehicle, an operating medium, the control logic, and the human. The human, of course, must be considered both as a part of the control logic and as a system user. In each, there are various alternatives that might be listed in the light of present and future technology. The interaction of these four elements is highly critical.

Then operating rules will be generated. These are to be the bases by which may be specified the characteristics of individual transportation in terms of the performance criteria and system variables. These are the theoretical expressions of a generalized systems concept. The operating rules may be expressed as a set of equations and the variables could then be related in such a way as to meet the defined performance criteria. Thus, it could be that a description of individual transportation would be stated as a set of mathematical functions.

Once this generalized framework for individual transportation has been developed, any combination of vehicle, operating medium, and control logic can be tested. Ultimately, one or several of the possible combinations that best satisfy these equations will be chosen for more intensive analysis. This is not a simple straightforward procedure; there must be feedback and interaction among the various steps. The generated operating rules and the several alternative solutions will point to the competence of the original performance requirements. Conversely, the continuous refinement of this performance statement must be accurately reflected in the operating rules.

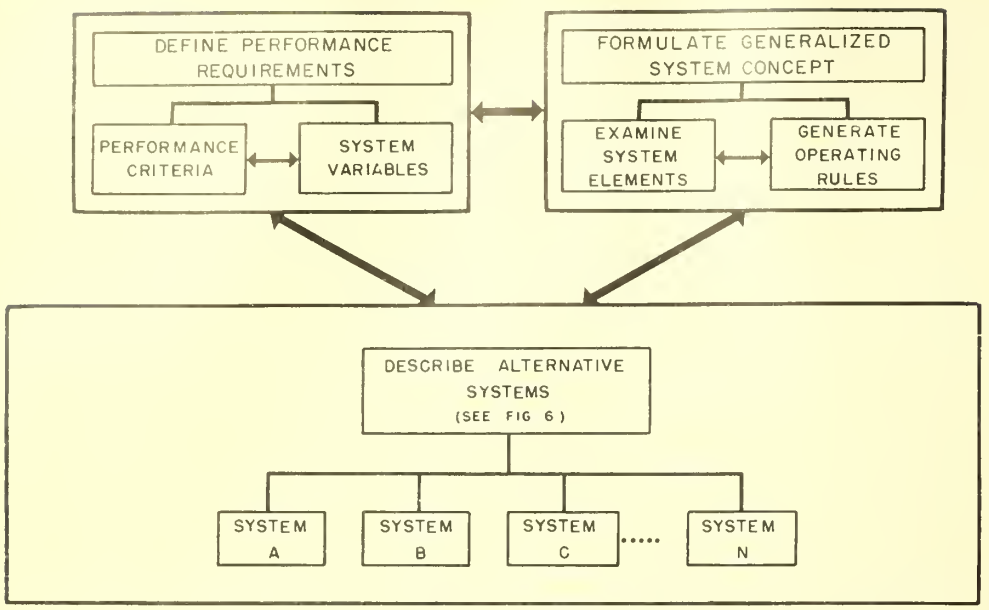

Figure 5.—Outline of the systems analysis for individual transportation.

Because this feedback process is of such vital importance to a systems analysis, the analysis can become extremely complex, particularly when dealing with a system so encompassing as individual transportation. Moreover, the systems analysis will form the basis for the entire long-range research and development program. Hence, it is evident that the systems analysis should be done as a single operation in order to provide a solid framework around which all succeeding steps can be taken.

Precaution must be taken to avoid initial error, because any concept adopted and implemented would undoubtedly involve a significant portion of our national effort. To prevent hasty judgment and preselection of the most obvious (or any other) form of individual transportation as the "only" solution, it is desirable to explore all alternative concepts that could possibly meet the same objectives. Therefore, the systems analysis must investigate feasibility from broad viewpoints and determine the detailed technical concepts worthy of further research, development, and evaluation. It will define various alternative system concepts, bring them into a common analytical frame of reference, and compare their relative effectiveness.

One result of the systems analysis, incidentally, might be to indicate that modification of the existing system is the optimum way to proceed in the research and development phase. But, if this is the case, it will be clearly established that other alternatives have been investigated and rejected, and the reasons for such rejection will be detailed. Thus, it has been shown that by a systems analysis of the criteria, the variables, and the components, one arrives at a theoretical expression of individual transportation. For the first time there will be a comprehensive model of a major transportation medium from which to select optimum solutions. This generalized concept will permit the preliminary testing of many individual system combinations and the selection of those that best satisfy the general expressions.

The Alternative Systems

It has been shown that the three interdependent operations illustrated in figure 5 comprise a theoretical systems analysis. A general procedure was outlined for defining the performance requirements and formulating a generalized system. Now the third operation, the description of alternative systems will be discussed. However, these three operations are interdependent and must therefore be undertaken as a carefully coordinated effort.

By way of definition, an alternative system is the combination of operating components that will accomplish a given objective in an acceptable manner. In this case, describing an alternative system means proposing a complete solution to the problem of improving individual transportation. By this process, several alternative systems, not just one, may evolve. But, a properly conducted systems analysis will produce the minimum number of maximum efficiency systems. Each system will be complete, and each system can be accurately described.

These alternative systems, as shown in figure 6, obviously cannot be named at this time. However, they might include such systems as the oft referred to but so far vaguely described "automated highway." One system might be a conveyor belt highway and another might be some form of pneumatic tube transport. Or, with visions on the horizon of the possibilities of the future, one system may utilize airborne vehicles guided by laser beams and propelled by the energy received from the lasers. Other systems concepts will comp some n number of alternatives.

As shown in figure 6, there will be a dese tion of the operating characteristics of e alternative system, which will describe l the components within that system inter The subsystems of which any system must comprised will be described from all aspe In this description, consideration of the e ronment will include analyzing the featu of the areas through which the system it will operate, such as the land areas of the b ness district, the city, the suburbs, and rural areas. It will also include solutions those problems of entrance, exit, and stor of vehicles within the system. And, of cou it will describe the effects of environme problems such as weather.

The subsystems will be described from viewpoints of the various user groups. these, the largest group will consist of th who are desirous of improved personal tra portation. But full consideration will also given to that group of individuals who w to join with others to share improved n transportation; and to a third group, wh will include those individuals who desire improve the movement of freight. There also be a description of the probability of ceptance of each given alternative syst This may well be based on a description its comparability with the present highw system or it may use some other datum evaluation. It will, of course, include a c plete description of the readjustment ne sary in our economy to accept the new proposed systems.

Figure 6 also shows the four categories basic components in any transporta system: the vehicle, the operating medi the control logic, and the human. C interconnection must exist among all fou these component categories. It is not feas to develop one of the components without consideration of the others.

The vehicle will be considered as a conta for that which is to be transported. In e alternative system the vehicle will be compr of some combination of power sources propulsion techniques, as shown in figur This 2 × 2 matrix shows some existing familiar vehicles. But many other vehi could be placed in the matrix. The powered, externally propelled vehicle not exist at present. However, a syst analysis would generically describe new unique vehicles that in concept may n use of such techniques, and it is enti possible that such a vehicle could be veloped. The systems analysis will desc the useful characteristics of the prop vehicles and contrast them with undesir characteristics such as air pollution. F such descriptive comparisons of vehicles, usefulness of each as a system compo will become evident. These will be general rather than detailed, technical description

The operating medium of any alterna system in this systems analysis will described by its features as a "highw The term highway is used in the sense of AASHO definition, "A general term deno

public way for purposes of vehicular travel, cluding the entire area within the right-of-ay." As has been mentioned, the operating edium of an alternative system may well the conventional highway, with or without me modification. However, unconventional edia such as the ground pathway, various pes of structures, air, and many others need be considered and compared for the scription of alternative systems. Different bsystems possibly will require different edia to promote the most efficient movement traffic in each particular area. The de-ription of the operating medium will also dicate what provision must be made for such reign objects as pedestrians, animals, and bris.

The control logic of a system is that com-nation of techniques and devices employed regulate the operation of that system and outlined by the closed loop diagram shown figure 8. For each system, the analysis ll show what information needs to be ac-ired and how it will be obtained. The nception and design of the processing and alyzing equipment that will be necessary to nvert these data into operational decisions n then be described. The best means of mmunicating these decisions to the mechan-l equipment or the human, which will nslate them into the necessary action, can specified. The control logic loop is closed including the reaction, or feedback, which ll detect and correct the performance errors. he description of the control logic will also clude such things as the handling of non-nforming vehicles, failures in the logic itself in other parts of the system, and the ac-mmodation of personal emergencies.

It is also evident that as each alternative stem is described, the role of the human ust be considered both as an active system ement and as a rider. No analysis need sume the preconceived notion of complete tomation. The amazing ability of the man to accomplish perceptive and control sks has been previously pointed out. The pabilities and limitations of the human will studied and the results of these studies ll determine the areas where he may be ilized in guidance and control. The human ay well be the monitor of an automated stem; or the alternative system may be signed for human control that is to be tomatically monitored. The factors of fa-ue and vigilance also will be completely udied and described in each alternative stem.

The human as a rider in the system will ve great bearing on the acceptability of the stem. Studies of the tolerance of the human motion and to changes in motion in all rections will, of course, need to be made. so, it will be necessary to specify the train-g that will be required to fit the human to ch new system. These and other human aracteristics will bear equally with the other mponents on the utility and acceptability any system to the potential user.

Finally, a benefit-cost analysis will complete ch alternative system description. It is ident that in all areas it will not be possible

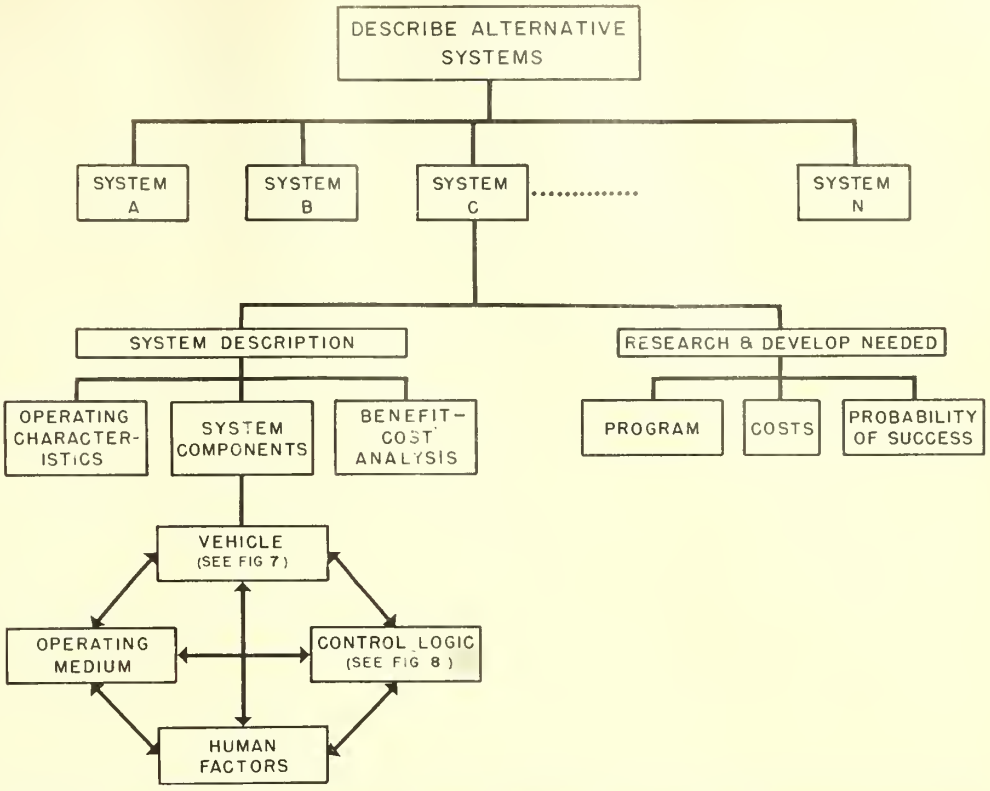

Figure 6.—Description of alternative systems including research and development needed.

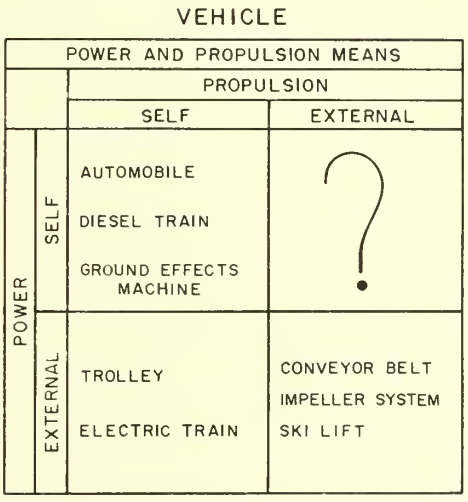

Figure 7.—Various combinations of vehicle power and propulsion.

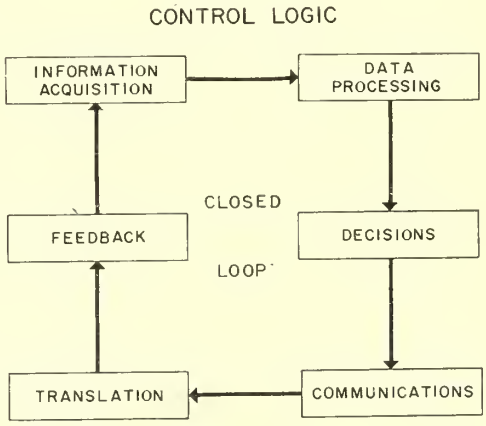

Figure 8.—Control logic considered as a closed loop process.

to indicate these items in terms of dollars. However, where it is not possible to estimate an exact cost in one alternative, the same base of comparison will be extended to the other alternatives. Of principal importance, however, is that the comparative cost between alternative systems be properly made. These costs, of course, can be categorized as initial, operating, and maintenance, and the benefits can be identifiable in each system. Those benefits which cannot be expressed in dollars and cents will be expressed in such terms as comfort and convenience to the potential user.

Thus out of the systems analysis will come a description of alternative types of systems that meet the requirements for individual transportation. In addition, the systems analysis will define the research and development needed to produce a prototype, as shown in figure 6. Included will be a complete description of the research and development program needed to determine the feasibility of and the design requirements for a prototype system. In addition, the cost of the research will be included and the probability of success in producing a prototype for testing will be estimated. It is recognized that some aspects of such a program will be only broadly identified in the systems analysis phase but will be detailed as research and development proceeds.

From this discussion, it can be seen that the systems analysis is a logical approach to the challenge of today—namely, to lay the foundations for the rational evolution of individual transportation systems of the future.

Comparisons of Empty and Gross Weights of Commercial Vehicles

BY THE BUREAU OF PUBLIC ROADS

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Office of Research and Development
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Office of Highway Planning

The need for a uniform weight classification base for commercial vehicles and the possibility of determining such a base from available information are described in this article. Because more adequate descriptions of commercial vehicles would permit better research and planning for the highways now being planned and built for the more than 100 million vehicles expected by 1972, an analysis has been made of available information.

Comparisons were made of data samples on commercial vehicles taken from the 1957 and 1961 loadometer studies and from special California vehicle records. Each sample group of data was satisfactorily representative of the total available information and correlations from selected groups of data were made by empty weights and by registered gross weights of vehicles.

The tabulations and the accompanying graphic materials are expected to be useful as guides in the solution of many vehicle classification problems. This analysis revealed that it would be very difficult, if not impossible, to develop a usable set of weight relationships from present registration data. However, the data considered in this study tend to give each other mutual support and the results of the 1957 loadometer study remain generally applicable.

Introduction

A SIGNIFICANT portion of highway research is dependent on the basic data that can be obtained on the numbers and types of motor vehicles that are, or are likely to be, in use. It is somewhat of an oddity that in this Nation of highly developed motor-vehicle mobility, one of the greatest single problems of highway research is the understanding, description, and cataloging of the numbers and kinds of vehicles in use for which highways must be provided.

There are nearly 80 million vehicles in the United States, and highways are now being planned and built for the more than 100 million expected 10 years from now. Yet, although each motor vehicle is required to be registered each year with a State motor vehicle department, it is possible to describe these 80 million vehicles in only the most general terms from the basic annual records. Although considerably more uniform information would be desirable on passenger vehicles the primary concern is the lack of uniform data on the types and weights of the truck fleet that at present is comprised of more than 12 million vehicles. The problems encountered are (1) the amount and quality of the data required and recorded on the annual registration application and on the registration certificate, and (2) the different weight bases used by the States for tax purposes. It often is not possible to combine, or to compare, the information on trucks registered in two neighboring States because the weight classification for tax purposes is entirely different. One State may register vehicles on the basis of the empty weight of the power unit, and another State may register its vehicles on the basis of the owner's declared maximum gross weight of vehicle and load. Data gathering is further complicated because the State using empty weight has no means for gross weight identification, and the State using gross weight frequently does not require the empty weight of the power unit for its records. Any significant comparison of the effect of the bases used for truck registration should include the numbers of vehicles registered by each method. The application of the three main weight classifications employed in State registration systems to the truck fleets in 1931, 1951, and 1961 is shown in figure 1. During the period from 1931 to 1961 truck registrations increased nearly fourfold, from 3.6 million to 12.3 million.[2]

Disparity in the methods of registration required has also been disappearing since 1931 when 26 States registered about 945,000 trucks on the basis of the manufacturers' rated capacities; 13 States registered approximately 1.6 million trucks on the basis of empty weight, and the remaining 10 States registered 1.1 million trucks on the basis of declared gross vehicle weight. By 1961 only Alabama retained the requirement for registration on the basis of manufacturers' rated capacity—239,000 trucks were registered. The rest of the States required trucks to be registered either by empty weight or by some form of declared gross weight. A total of 3.3 million trucks was registered in 14 States by empty weight, and 8.8 million trucks were registered in 36 States by declared gross weight. Except for the small 2-axle truck, commonly appearing as a pick up or panel vehicle and having characteristics similar to a passenger car, the many different types and sizes of trucks and combinations that compound the problems of classification and taxation are shown in silhouette in figure 2.

In this article several samples of data th relate vehicle empty weights and register gross weights have been compared in ord to establish a set of usable weight correlatio by visual vehicle classes. The resultant weig comparisons are given in tabular form a both the vehicle distributions and their pe centage counterparts shown. These compa sons (tables 11–17) provide an addition classification tool for research and planni activities.

The research covered by this report w have many uses, important to the Federal a the State governments. The data present can be used as an aid in the analysis of t application and equitability of road-user tax and they are expected to enhance the effectiv ness of administration of motor-vehicle t laws. They will be useful in determining t probable effects of legislation proposed, a they also will be of value to those concern with highway planning, and to industry materials, product, and market research.

Vehicle Classification Studies

One of the early efforts to count and classif commercial motor vehicles was a comprehe sive study of registrations and fees reported *The Taxation of Motor Vehicles in 1932*, b G. P. St. Clair, PUBLIC ROADS, vol. 15, No. Oct. 1934, pp. 185–214. Information for th study was compiled by the Bureau of Publ Roads from State and local motor-vehic records and from questionnaires that request data on vehicles and taxes in considerab

[1] Presented at the 42d annual meeting of the Highway Research Board, Washington, D.C., January 1963.

[2] Data for the 1931 and 1951 comparisons were collected from 48 States and the District of Columbia. Information from Alaska and Hawaii have been included in 1961 figures.

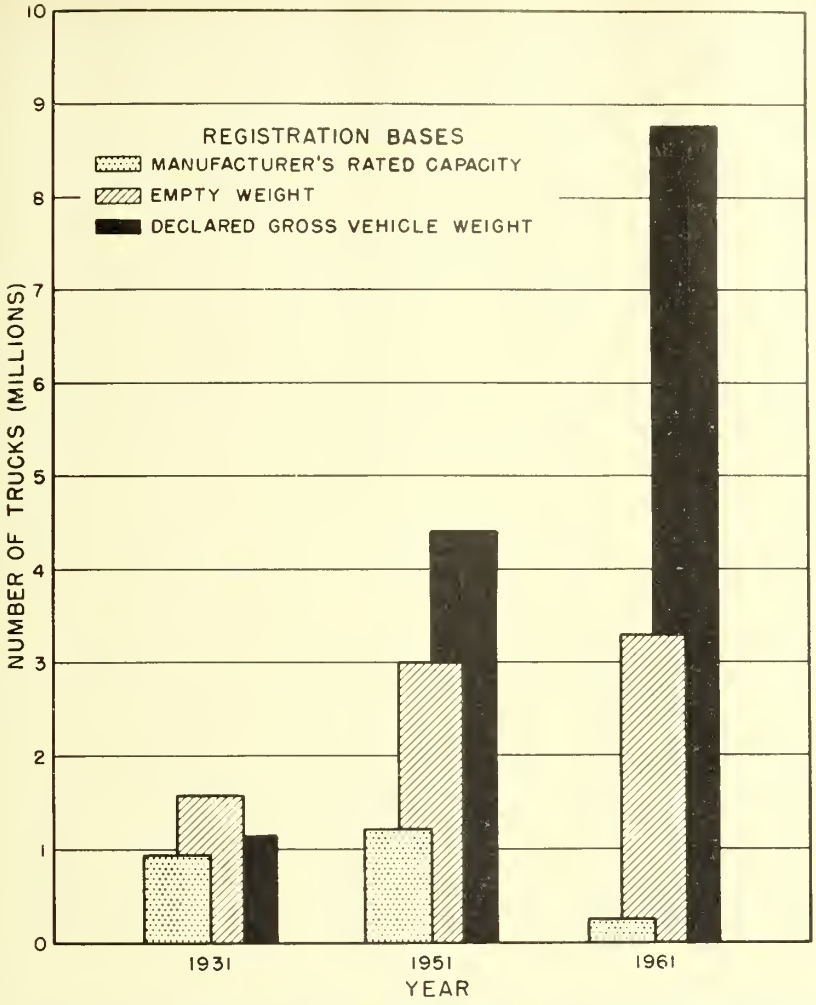

Figure 1.—Number of trucks and combinations registered in 1931, 1951, and 1961, segregated by registration base. Data for 1931 and 1951 are comparable but 1961 data include registrations in Alaska and Hawaii in the empty weight bar.

detail. Another study, known as the Nationwide Truck and Bus Inventory, was begun in 1940 by the Bureau of Public Roads in cooperation with the States. Although the work was eventually completed, it was expensive, and it used manufacturers' rated capacities as a uniform measure of truck weight. Since the use of that classification was rapidly waning, the study had limited value for comparing current vehicle classification data, and the results of the study have not been published.

The next major vehicle classification study was made by the Bureau of Public Roads, in cooperation with the States, to provide basic information for the highway cost allocation study that was required by Section 210 of the Highway Revenue Act of 1956. The findings of this classification study were included in the comprehensive series of highway cost allocation study reports made to the Congress, and also were published in 1960 by the Bureau of Public Roads as the *Classification of Motor Vehicles, 1956–57*. This study is the most recent inventory of highway rolling stock, and it will be referred to in this report as the classification study.

When the classification study was undertaken, an effort was made by Public Roads and State authorities to obtain the needed data in each of the States. Intensive reviews were made of the existing registration records, special questions were added to some motor-vehicle registration application forms for the following year, and special questionnaires were mailed to vehicle owners by many States in an effort to obtain information to supplement the data in the registration files. A valuable lesson was learned during this study. The motor-vehicle data needed for highway research were unavailable from any public source in a usable form. Even if it had been possible to obtain a complete summary and analysis of the vehicle records of each State, the data obtained would have been so lacking in uniformity that it would have been impossible, with the knowledge then available, to combine them into a workable, usable body of data for use in research. One result of these findings is the cooperative effort of the States and Public Roads to develop standard vehicle descriptions and information that will be useful to both government and industry. As a result of this effort, substantial progress is being made under the auspices of the American Association of Motor Vehicle Administrators.

Many differences existed in the registration requirements and records of the States but the one that posed the greatest problem was the requirement of several States for registration of vehicles on the basis of empty weight or on variations of gross and empty weights. Most States registered and recorded vehicles on the basis of the owners' declared gross weight (the weight of the vehicle, fully equipped and ready for service, plus the maximum load to be carried).

When it is necessary, in studies of motor vehicles or motor-vehicle revenues, to bring the basic motor-vehicle data of all States into uniformity, a relationship must be established between the bases and all of the data must be converted to a uniform structure for analysis.

To properly analyze the composition of the vehicle fleet, an understanding of the factors affecting the selection of the vehicles in use is necessary. Tax structures, terrain, kind of goods transported, and literally dozens of factors affect owners' vehicle selections. Some carriers may elect to buy lightweight power equipment to perform the same job that is done by another carrier with heavier and costlier power units. The lighter power units would depreciate more rapidly but, because of other factors, they might provide lower overall operation cost. The subject of vehicle ownership and operating costs is discussed in considerable detail in the report *Line-Haul Trucking Costs in Relation to Vehicle Gross Weights*, by Hoy Stevens, Highway Research Board Bulletin 301, 1961.

Sources of Data for Weight Comparisons

1957 traffic and loadometer data

During the course of the extensive 1957 motor-vehicle traffic counting, classification, and loadometer operations, approximately 600,000 vehicles were weighed, and data concerning empty weight, registered weight, make, body, axle arrangement, and other items on vehicle classification and operation were obtained. More than 150,000 commercial vehicles, for which weight data were complete, were selected from the group of 600,000 for special study to relate empty and registered gross vehicle weights. Gross vehicle weight was available from the registration certificates for only vehicles registered on that basis, but it is believed that a good representative sample was obtained because States using this basis were very well distributed geographically. In this article, the data concerning the 150,000 commercial vehicles is referred to as the "1957 loadometer data." Information from more recent weighing studies and spot vehicle classification counts made by the States have been added to the 1957 loadometer data. The locations of the weighing stations were selected with the objective of making the data collected from them representative of the vehicles being used in that area.

1961 loadometer data

Rather than wait until the 1961 loadometer study had been completed and the complete record of weighings was available for use, a special group of data was collected from a limited sample of vehicles throughout the United States. This sample was obtained as

Table 1.—Trucks and combinations, observed during 1957 and 1961 loadometer studies, grouped by number of axles and by registered gross vehicle weights [1]

Registered gross vehicle weight	Single-unit trucks				Combinations consisting of—											
					Tractor and semitrailer						Truck and full trailer				Tractor, semitrailer and full trailer	
	2-axles		3-axles		3-axles (2-S1)		4-axles (2-S2)		5-axles (3-S2)		3-axles (2-1)		5-axles (3-2)		5-axles (2-S1-2)	
Pounds	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*
0-3,999																
4,000-4,999	49,279	36.0														
5,000-5,999	26,846	19.6														
6,000-7,999	12,767	9.3														
8,000-9,999	6,637	4.9														
10,000-11,999	5,456	4.0														
12,000-13,999	4,560	3.3														
14,000-15,999	4,236	3.1														
16,000-17,999	6,855	5.0	152	2.1	2											
18,000-19,999	4,431	3.2	47	0.6	106	1.6					28	9.2				
20,000-21,999	5,761	4.2	65	0.9	77	1.2					14	4.6				
22,000-23,999	3,000	2.2	106	1.5	93	1.5	29	0.3			17	5.5				
24,000-25,999	4,732	3.5	193	2.6	241	3.8	35	0.4			14	4.6				
26,000-27,999	1,153	0.8	205	2.8	127	2.0	22	0.2			16	5.2				
28,000-29,999	294	0.2	214	2.9	187	3.0	11	0.1			14	4.6				
30,000-31,999	520	0.4	322	4.4	394	6.3	38	0.4			14	4.6				
32,000-35,999	103	0.1	708	9.6	1,040	16.5	47	0.5			38	12.4				
36,000-39,999	103	0.1	1,174	16.0	987	15.7	101	1.1			53	17.3				
40,000-44,999	97	0.1	1,657	22.5	2,188	34.8	280	3.2			86	28.1				
45,000-49,999	41		2,273	30.9	301	4.8	361	4.1	191	3.3	12	3.9				
50,000-54,999	21		233	3.2	376	6.0	1,843	20.8	151	2.6					1	1.5
55,000-59,999	9				66	1.0	4,061	45.9	192	3.3			17	2.4		
60,000-64,999	56				104	1.7	1,737	19.6	1,070	18.3			5	0.7	2	2.9
65,000-69,999					6	0.1	261	3.0	1,216	20.9			42	5.9	4	5.9
70,000-74,999							34	0.4	2,595	44.5			311	43.5	28	41.2
75,000-79,999									416	7.1			319	44.6	30	44.1
80,000 and over													21	2.9	3	4.4
TOTAL	136,957	100.0	7,349	100.0	6,295	100.0	8,860	100.0	5,831	100.0	306	100.0	715	100.0	68	100.0

[1] Data from 1957 and 1961 special, field-weighing reports are combined in this table. The portion of the table boxed by heavy lines represents 90 percent or more of the vehicles in each vehicle type.

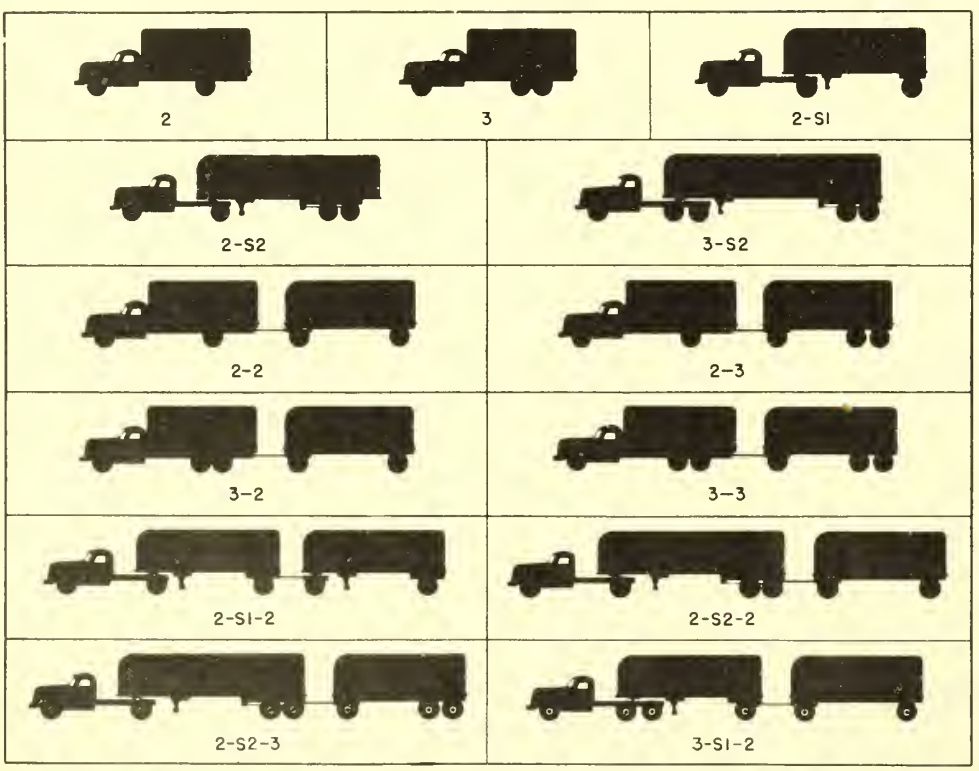

Figure 2.—Commercial vehicle types as designated by code based on axle arrangement.

a part of the regular loadometer study, but was collected at the first station or first two stations operated in each State at the beginning of the weighing operations. The study instructions stipulated that vehicles were to be weighed at each station until at least 10 loaded and 10 empty vehicles of each visual type, as shown in figure 2, had been observed.

A field crew member was assigned to interview each driver and to obtain registration card information while the vehicle was being weighed by other members of the crew. These data were placed on punch cards, which were forwarded to the Washington office of the Bureau of Public Roads. In order to check the accuracy of the sample, Public Roads sent the record of each of these vehicles to the State in which it was registered to be verified against the registration file. It is believed that this check eliminated many of the inconsistencies, which might otherwise have gone undetected, and that data for the resultant group of vehicles identified in this article as the "1961 loadometer data" have a relatively high degree of accuracy. Although the sample was not expanded, a comparison of the data with those obtained from other sources showed the information to be representative in all major weight cells. The usable sample from the 1961 loadometer data totaled approximately 14,000 vehicles, and the information gathered included empty and gross

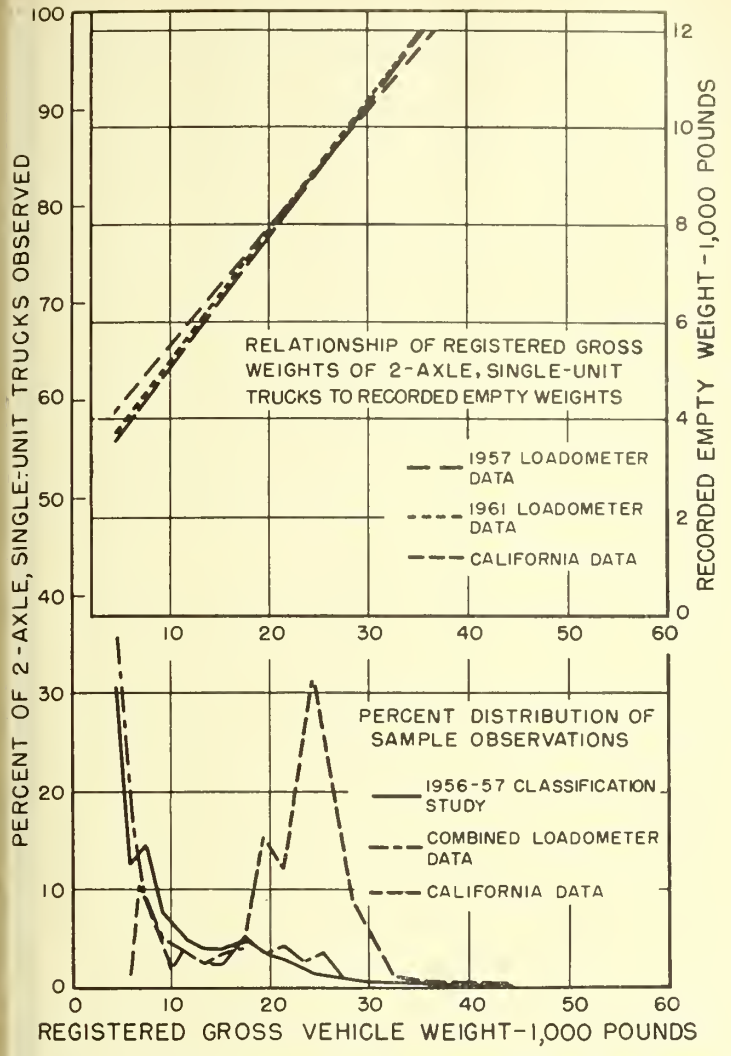
RELATIONSHIP OF REGISTERED GROSS WEIGHTS OF 2-AXLE, SINGLE-UNIT TRUCKS TO RECORDED EMPTY WEIGHTS
1957 LOADOMETER DATA
1961 LOADOMETER DATA
CALIFORNIA DATA
PERCENT DISTRIBUTION OF SAMPLE OBSERVATIONS
1956-57 CLASSIFICATION STUDY
COMBINED LOADOMETER DATA
CALIFORNIA DATA
PERCENT OF 2-AXLE, SINGLE-UNIT TRUCKS OBSERVED
RECORDED EMPTY WEIGHT-1,000 POUNDS
REGISTERED GROSS VEHICLE WEIGHT-1,000 POUNDS

Figure 3.—Empty to gross weight relationships and relative distribution of 2-axle, single-unit trucks.

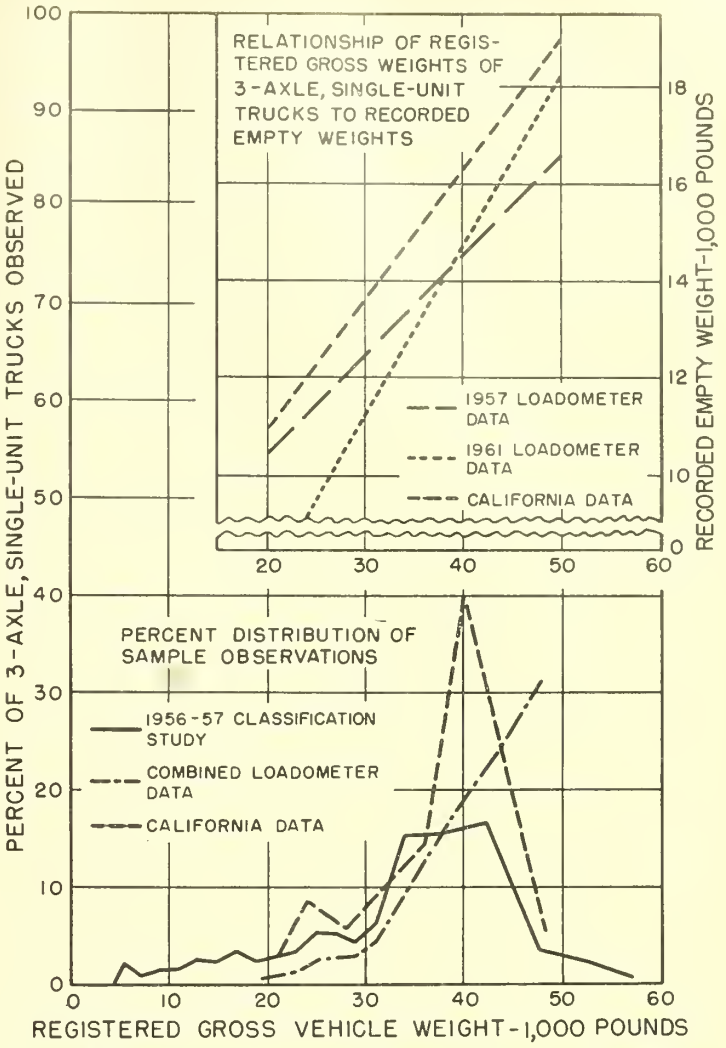
RELATIONSHIP OF REGISTERED GROSS WEIGHTS OF 3-AXLE, SINGLE-UNIT TRUCKS TO RECORDED EMPTY WEIGHTS
1957 LOADOMETER DATA
1961 LOADOMETER DATA
CALIFORNIA DATA
PERCENT DISTRIBUTION OF SAMPLE OBSERVATIONS
1956-57 CLASSIFICATION STUDY
COMBINED LOADOMETER DATA
CALIFORNIA DATA
PERCENT OF 3-AXLE, SINGLE-UNIT TRUCKS OBSERVED
RECORDED EMPTY WEIGHT-1,000 POUNDS
REGISTERED GROSS VEHICLE WEIGHT-1,000 POUNDS

Figure 4.—Empty to gross weight relationships and relative distribution of 3-axle, single-unit trucks.

weights, vehicle type, number of axles, body type, class of use, some information on fuel used, year model, make of vehicle, and commodity carried. Only the information that applies specifically to weight comparisons has been summarized here. Processing of the remaining data is in progress and, if these data are found to be representative, they will be used in other studies.

Some unexplained differences were noted in a comparison of the 1957 and 1961 loadometer data. These differences probably were caused by the highway system coverage and the distribution of the loadometer stations. Because of the scope and purpose of the 1957 loadometer study, more urban stations were included and a greater coverage of secondary and local road systems was obtained. The 1961 loadometer data, however, are more indicative of the type of vehicles used on main rural highways.

California data

The third group of data used in preparation of this article was obtained from the State of California for vehicles registered under the Uniform Proration Compact. California maintains an excellent file on motor-vehicle fleets that are registered in other States on different registration bases and that are operated in California under the Proration Compact. Uniform empty and gross weight data and other vehicle information were available for these vehicles. The California authorities permitted the authors to use the information and provided much assistance in interpreting it. This availability of another source of data was an important factor in the decision to present this study.

Unlike the truck samples obtained in the loadometer surveys, the California data represented principally over-the-road fleets from the Western States. The records included the declared gross vehicle weight of the vehicle or combination; the empty weight of the power unit; and the type of carrier, make, year model, and number of axles; and the type of motor fuel used. Data on approximately 8,000 vehicles were supplied by the State, and information on 6,700 has been used in the comparisons in this article. Information on approximately 1,300 vehicles could not be included in the study because one or more of the basic weight factors had not been included in the reports to the State.

Data from other sources

The State motor-vehicle registration authorities make their annual registration counts, by vehicle type, available to the Bureau of Public Roads and other interested groups. These data are consolidated in Public Roads tables MV–1 through MV–11 [3] for use by government transportation and planning authorities, industry marketing groups, and private individuals. A few States prepare special tabulations on commercial vehicles by weight classes for their own uses, and copies of these have been supplied to the Bureau of Public Roads for studies of vehicle characteristics, distribution, and use.

Discussion of Data

Registered gross weights by vehicle types

A summary is shown in table 1 for the vehicles registered on a gross weight basis for which empty weights were available; these data were obtained in the 1957 and 1961 loadometer surveys. Numbers and percentages of vehicles of each type are given by registered gross weights. Heavy lines in the table enclose data for approximately 90 percent of the vehicles in each visual type. The extremes, representing approximately 10 percent of the vehicles, are "fenced out" above and below the main group. Thus a visual comparison

[3] Bureau of Public Roads tables MV–1 through MV–11, *Highway Statistics*, issued annually.

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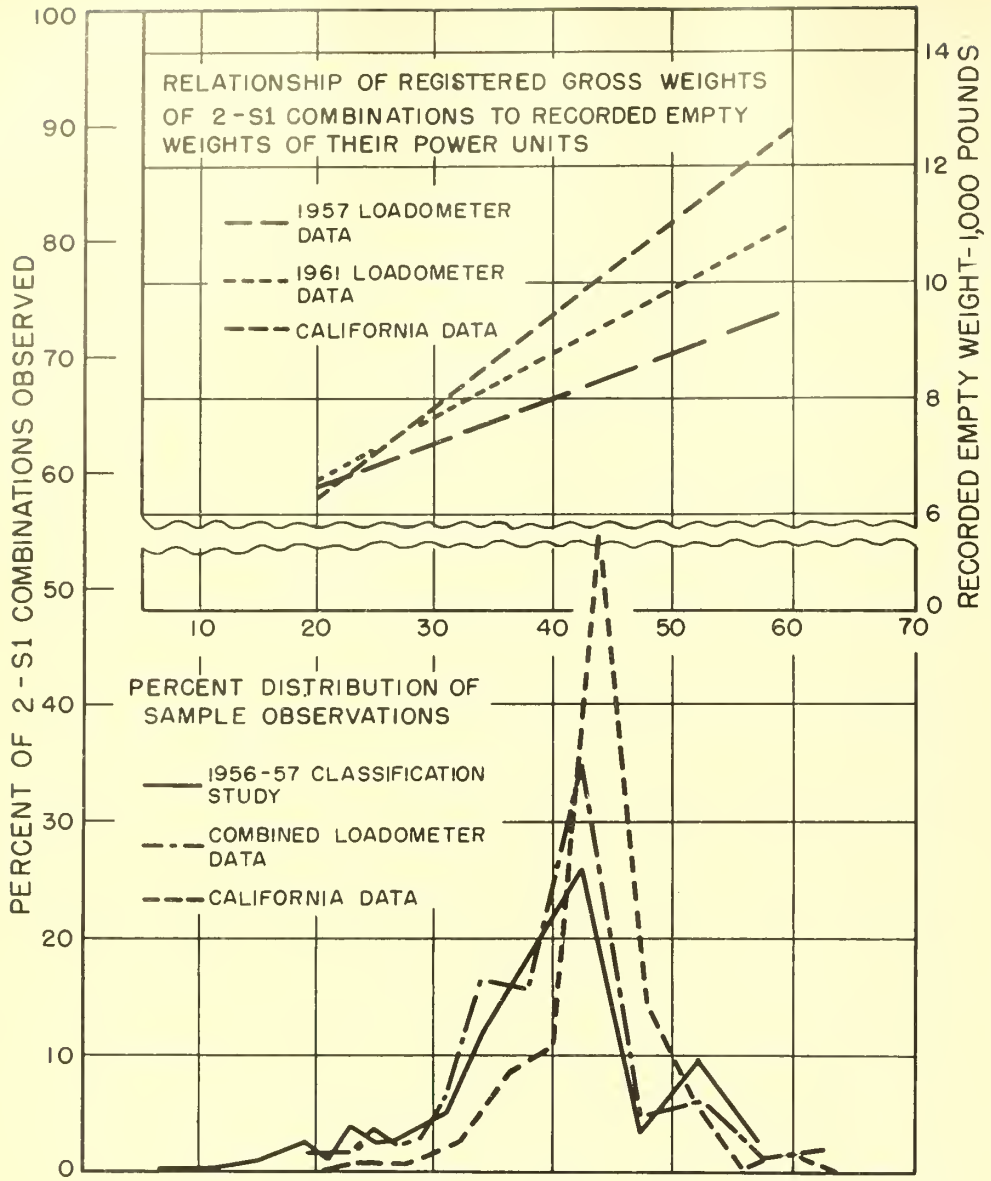
RELATIONSHIP OF REGISTERED GROSS WEIGHTS OF 2-S1 COMBINATIONS TO RECORDED EMPTY WEIGHTS OF THEIR POWER UNITS
1957 LOADOMETER DATA
1961 LOADOMETER DATA
CALIFORNIA DATA
PERCENT DISTRIBUTION OF SAMPLE OBSERVATIONS
1956-57 CLASSIFICATION STUDY
COMBINED LOADOMETER DATA
CALIFORNIA DATA
PERCENT OF 2-S1 COMBINATIONS OBSERVED
RECORDED EMPTY WEIGHT-1,000 POUNDS
REGISTERED GROSS VEHICLE WEIGHT-1,000 POUNDS

Figure 5.—Empty to gross weight relationships and relative distribution of 3-axle, tractor-semitrailer combinations (2-S1).

can be made of the total range of the data. This comparison shows the approximately 90-percent spread of gross weights for each of the vehicle types, and it illustrates that as the vehicles became larger the gross weight range was smaller. Registered gross weights for each vehicle type, however, overlap the weights for both adjacent vehicle types.

The 1961 loadometer data presented in this study for the 2-axle trucks cannot be separated into 4-tire and 6-tire classes. Other sources[4] have shown however that, taken as separate groups, the 2-axle, 4-tire class would show a rapid diminution of numbers over 8,000 pounds and, with the greater load flexibility permitted by additional tires, the 2-axle, 6-tire class would peak at about 12,000 to 18,000 pounds and would taper off slowly in numbers at approximately 28,000 pounds. Within the enclosed area of the table, the data for successive vehicle types form a group of steps to the larger gross weights.

[4] *Classification of Motor Vehicles, 1956–57*, Bureau of Public Roads, U.S. Department of Commerce, 1960.

Comparison of 1957 and 1961 loadometer data and California data

Table 2 shows the California data by registered gross weights and by visual vehicle types. The heavy lines used, as in table 1, enclose approximately 90 percent of the vehicles in each type. A comparison of the vehicle distributions from the loadometer weighings shown in table 1 with those obtained from the California data included in table 2 reveals considerable disparity in the information from the two sources. Because vehicles represented in the California data were used principally in intercity service, much less dispersion in gross weights was noted in these data than in the information obtained from the loadometer studies.

Frequency distributions and least squares comparisons of empty to gross weights are shown in figures 3–9 for the main visual type of vehicles. The California data, represented by the medium-length dash least squares line in the upper panels of these figures, with certain exceptions, showed that the average empty weights of vehicles in relation to given gross weights were higher than the empty weight to gross weight relations recorded by loadometer data. A similar empty weight relationship was not recorded for the 3–S2 vehicle combinations; the slope of the line for the 1961 loadometer data, shown in figure 7, suggests the effect of too small a sample. However, this relationship of the empty to gross weight probably is not entirely accurate as the Public Roads' vehicle classification counts indicate that use of the 3–S2 vehicle combinations has become more widespread geographically than in 1957, and therefore the relationship of empty to gross weight could have been different than shown by the 1961 loadometer data.

As shown in figure 8, an exception to the higher empty weights in relation to gross weights was recorded in the 1957 loadometer data, which included information on an unusually large number of 3–2 truck-trailer combinations registered at 50,000 to 55,000 pounds gross combination weight and reported as having empty weights of more than 16,000 pounds for the truck alone. Such a reported distribution of so many 3–2 combinations at 55,000 pounds in 1957 was not normal because in the classification study nearly 97 percent of the 3–2 combinations were reported to have been registered at more than 60,000 pounds gross combination weight.

A percentage comparison of the gross weight distribution of combined 1957 and 1961 loadometer data and of the California data with the nationwide gross weight distribution of all vehicles of each type reported in the 1956–57 classification study is given in the bottom panels of figures 3–6. As shown in figure 3, the loadometer data distribution by gross weight was close to that for the classification study. This close relationship implies that the gross weights for vehicles sampled in the loadometer studies were relatively proportional to the gross weights for all such vehicles registered. But, as stated earlier, the California data, consisting largely of registrations of over-the-road 2-axle, 6-tire vehicles showed a much larger sample for vehicles having 18,000 to 26,000 pounds gross weights. The 2-axle classification given in figure 3 included both the 2-axle, 4-tire and the 2-axle, 6-tire vehicles. Nationwide more than 90 percent of the 2-axle, 4-tire vehicles were registered for gross weights under 8,000 pounds. More than 67 percent of the 2-axle, 6-tire trucks were registered for gross weights in excess of 12,000 pounds, and nearly 47 percent were registered for gross weights in excess of 16,000 pounds.

Figures 4 through 9 show that the gross weights of the sampled vehicles in the loadometer studies follow closely the gross weight distributions of the vehicle population. Gross weight comparisons for information from the classification study have not been included in figures 7 through 9 for the 3–S2, 3–2, and the

2–S1–2 vehicle combinations because these vehicles generally are registered for the State maximum permitted gross weights of over 50,000 pounds and their registrations were shown in the classification study in that maximum weight class.

Combined loadometer data

In figure 10, straight lines illustrate the empty to gross weight relationships obtained by the least squares method. The lines in this figure were based on the combined data from the loadometer surveys, and they provide a quick visual comparison of relationships for five vehicle types. The lines for the single-unit trucks follow a parallel course, they overlap in the gross weights from 22,000 to 32,000 pounds, and they are separated by about 1,500 pounds of empty weight. This greater empty weight is accounted for largely by the third axle in the 3-axle truck. The slope of these two lines is much steeper than the slope of the lines for the tractor power units, shown in combination as 2–S1, 2–S2, and 3–S2, because the payload carrying body is included in the empty weight for single-unit trucks but is not included for the combination vehicles. A considerable gross vehicle weight overlap is shown for the 2–S1 and 2–S2 combinations because of differences in size and weight requirements; some States require an additional axle to carry loads that can be carried by the 2–S1 combination in other States. Also, factors of terrain, power requirements, and types of loads carried are considered by operators in their choice of vehicles.

Comparison of 1957 and 1961 loadometer data

A percentage comparison of the distribution of gross weights of vehicles from the 1957 loadometer data with the distribution of the gross weights of vehicles from the 1961 loadometer data is shown in table 3. The 1957 study was designed to sample vehicles on all types of rural and urban highways as uniformly as possible, but the 1961 data were obtained to a larger extent at stations on main rural roads. The comparison shown in table 3 indicates that the traffic on main rural roads has a much greater concentration of heavy vehicles than the total traffic on all types of rural and urban highways.

Table 3 is complemented by table 4, which shows a distribution of the same vehicles by empty weights of the trucks and power units for the 1957 and 1961 loadometer surveys. The information in both of these tables shows that the empty and gross weights were consistently heavier in the 1961 loadometer data. In tables 3 and 4 the percentage distributions for each weight group, within each vehicle type, have been cumulated inversely as an additional check on the differences between the 1957 and 1961 loadometer data. At first glance it might appear that trucks and combinations have gotten heavier since 1957, and to some degree this may be true. However, evidence from continuing vehicle and classification counts have led the authors to conclude that most of the difference between

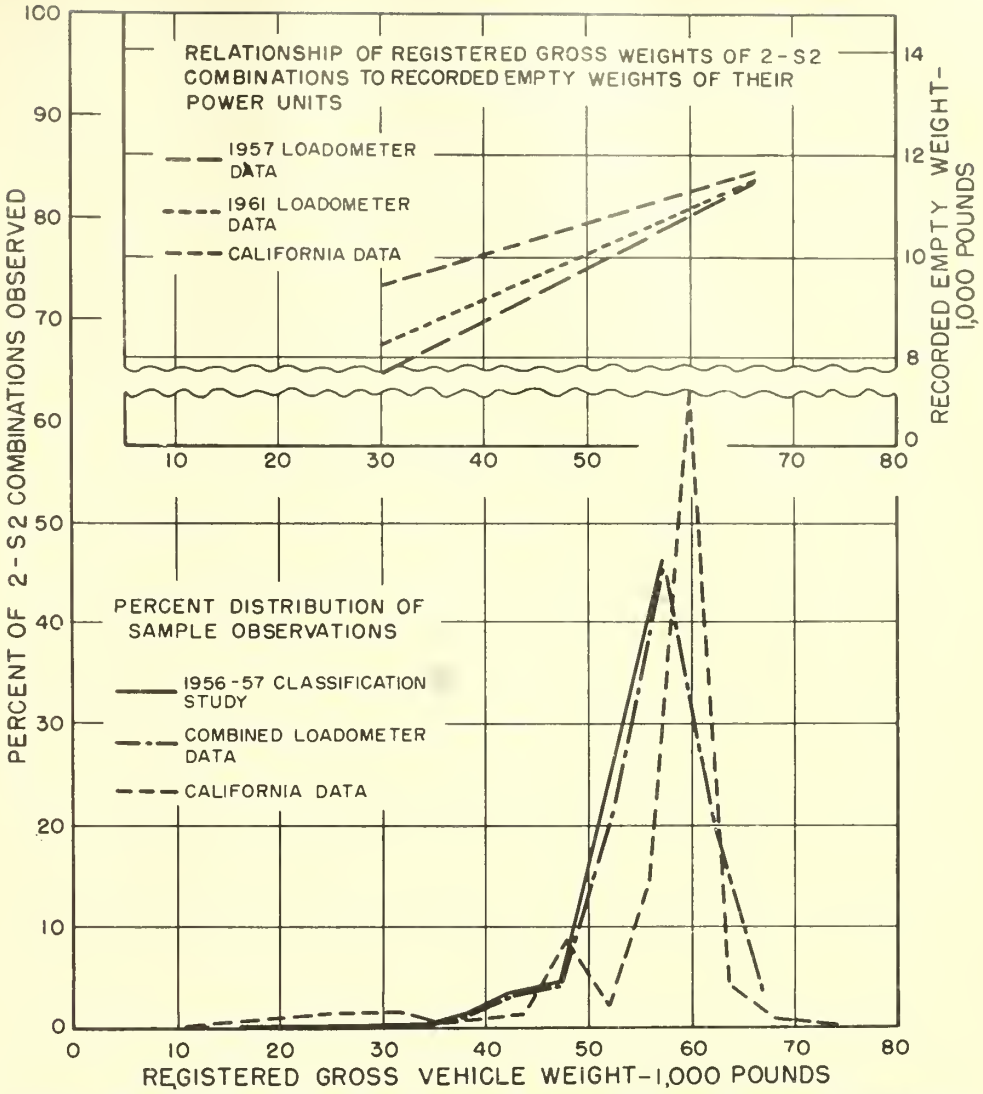
RELATIONSHIP OF REGISTERED GROSS WEIGHTS OF 2-S2 COMBINATIONS TO RECORDED EMPTY WEIGHTS OF THEIR POWER UNITS
1957 LOADOMETER DATA
1961 LOADOMETER DATA
CALIFORNIA DATA
RECORDED EMPTY WEIGHT-1,000 POUNDS
PERCENT OF 2-S2 COMBINATIONS OBSERVED
PERCENT DISTRIBUTION OF SAMPLE OBSERVATIONS
1956-57 CLASSIFICATION STUDY
COMBINED LOADOMETER DATA
CALIFORNIA DATA
REGISTERED GROSS VEHICLE WEIGHT-1,000 POUNDS

Figure 6.—Empty to gross weight relationships and relative distribution of 4-axle, tractor-semitrailer combinations (2-S2).

the two sets of data was caused by the difference in the size and scope of the samples.

To show a more complete cross-section of information on the three vehicle types given in tables 3 and 4, a set of two-way frequency distributions of empty weight to gross vehicle weight has been given for each of the three vehicle types separately for the 1957 and 1961 loadometer samples in tables 5–10. With the data arrayed in this manner it is possible to examine either the frequency distribution by empty weights of vehicles in a given class interval of registered gross weight, or the distribution by registered gross weights of vehicles in a given class interval of empty weight. Both numerical and percentage distributions are given, and heavy lines enclose approximately 90 percent of the vehicles in each empty weight group. When special consideration is given to the 90-percent portion of the sample in each table, the array of each vehicle type is much more compact. Although an appreciable number of vehicles are shown at the extremes, those having heavy empty weights and light gross weights and those having light empty weights and heavy gross weights constituted only a small proportion of all vehicles in that class. A large proportion of some vehicles of a given empty weight were concentrated in two or three gross-weight intervals.

Conversion tables

Tables 11 through 17 give the comparisons of empty weights to gross weights of the combined 1957 and 1961 loadometer data for seven of the most commonly used types of vehicles. Information on all the vehicles for which the weight data collected was usable for this article has been included in these tables. They give the numbers and percentages (horizontally) of the gross weight distribution of these vehicles. The numbers of vehicles that had unusual empty to gross weight relationships have been included even though they represent a very small percentage. The 166,000 vehicles that were classified by weights are representative of the national distribution of vehicles and their classification provides a tool for the solution of problems of weight conversions. These data will be useful for making revenue estimates, as well as being a working tool in many areas of market research.

The process of conversion is illustrated as follows. Assume that table 13 was considered appropriate, in a given situation, for con-

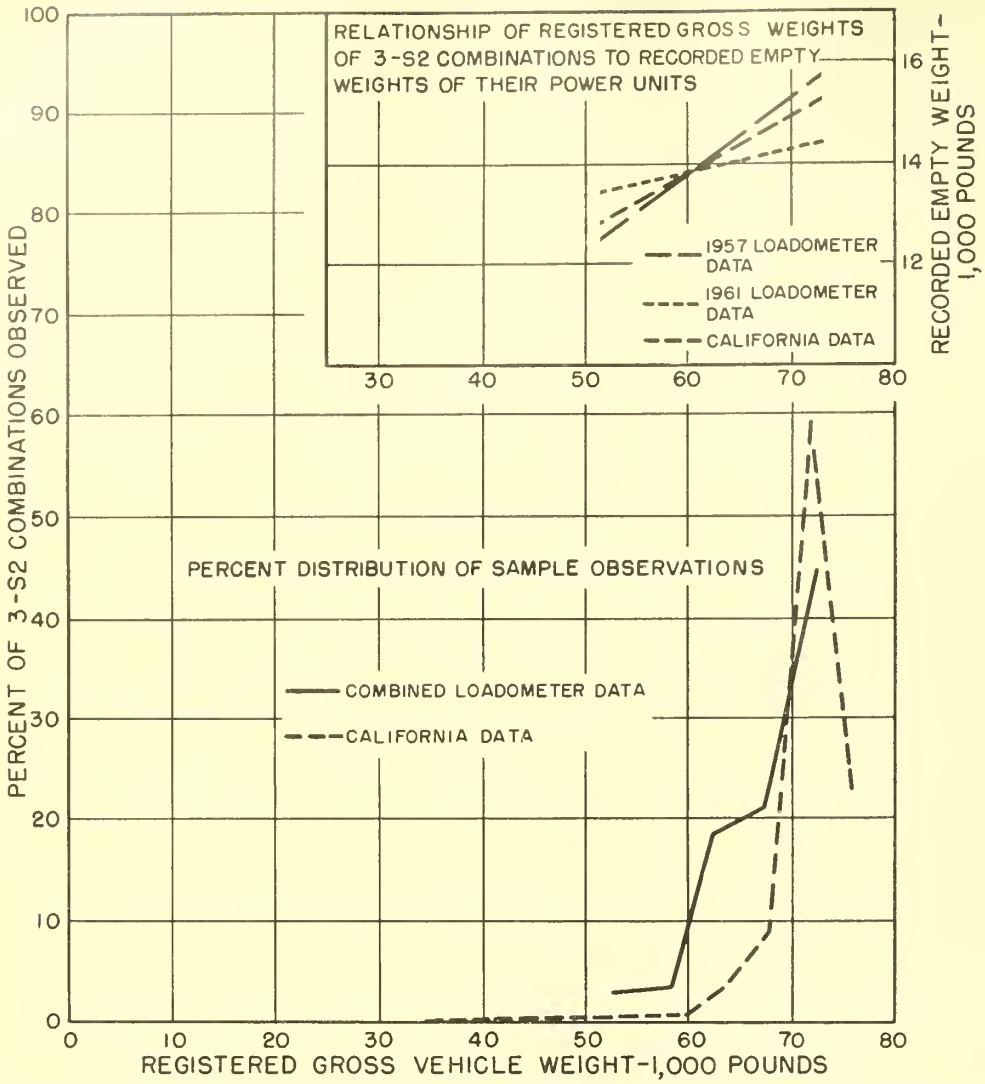
RELATIONSHIP OF REGISTERED GROSS WEIGHTS OF 3-S2 COMBINATIONS TO RECORDED EMPTY WEIGHTS OF THEIR POWER UNITS
1957 LOADOMETER DATA
1961 LOADOMETER DATA
CALIFORNIA DATA
RECORDED EMPTY WEIGHT-1,000 POUNDS
PERCENT DISTRIBUTION OF SAMPLE OBSERVATIONS
COMBINED LOADOMETER DATA
CALIFORNIA DATA
PERCENT OF 3-S2 COMBINATIONS OBSERVED
REGISTERED GROSS VEHICLE WEIGHT-1,000 POUNDS

Figure 7.—Empty to gross weight relationships and relative distribution of 5-axle, tractor-semitrailer combinations (3-S2).

verting 3-axle, tractor-semitrailer (2-S1) combinations registered by empty tractor weights into an array representing their probable distribution by registered gross weight of combination in a State requiring that method of registration. The number of vehicles in each class interval of empty weight should be multiplied by the corresponding horizontal percentages in table 13, and the numbers so obtained should be added vertically to obtain the distribution by registered gross weights. Conversely, a conversion from registered gross weight of combination to empty weight of tractor can be performed by distributing the number of vehicles in each gross weight class interval proportionate to the corresponding vertical distribution of vehicles by empty weights in table 13 and then adding the numbers so obtained horizontally.

Weight relationship of trailer and combination

In figure 11, a scattergram of the mean average empty weights and the lines of best fit reflect the approximate empty to gross weight relationship of tractors and semitrailers shown in the California data. Straight lines were computed for 1- and 2-axle, semitrailers and for the 2- and 3-axle tractor trucks used with them. The scattergram shows a wide range of empty weights of semitrailers in each type of tractor-semitrailer combination and at all gross weight levels. However, regardless of the type of combination, whether 2-S1, 2-S2, or 3-S2, even with substantial increases in gross combination weights, only moderate increases were noted in the semitrailer average empty weight. But for the tractor truck power units a much steeper gradation in empty weight in relation to gross weight is shown

Empty weight to gross weight ratios

Employing the power unit relationship used in figure 10 and the data from the semitrailer line in figure 11, empty weight to gross weight ratios shown in table 18 indicate that vehicle gross weights ranged from 1.2 times the empty weight at the low-weight interval of the smallest vehicle to a high of 2.8 at the high-weight interval for the larger vehicles. It may be of some significance that a vehicle type selected and registered at near the maximum weight of its class is capable of operating with the most favorable empty weight to gross weight ratio. The results for the upper gross weight limit of each vehicle type are similar for all five vehicle types.

Range of conversion

Figures 12 through 18 illustrate both the wide range of empty weights for each gross weight, and the range that contained approximately 90 percent of the vehicles. Although the 90-percent range eliminates the extremes, the band of weight comparison is still too wide to allow the use of a point of conversion. It would be very difficult, if not impossible, to develop a usable set of weight relationships that would permit a point, or even a narrow band, of weight conversion to be used for any purpose.

Conclusions

In general, data from the vehicle weight comparison series included in *Classification of Motor Vehicles, 1956–57*, the information from the 1957 and 1961 loadometer data, and the California data tend to give each other strong mutual support. Therefore, the results of the 1957 loadometer study remain generally applicable, and the study reported in this article is a further refinement of the data. In applying weight comparison factors from any of the data, however, some caution should be exercised to allow for the increasing trend toward use of diesel-powered vehicles and for the anticipated effects of any changes in vehicle size and weight laws.

The 1961 loadometer data and the California data have provided information that permits the addition of another large vehicle combination to the vehicle weight comparison series—the 2-S1-2. This combination was not covered in earlier studies. Additional investigation in this area is warranted, not only to obtain more data on the vehicle weight relationships, but also to keep the findings from these investigations up-to-date. Comprehensive studies of vehicles on a carefully tailored regional basis would provide information even more usable. In the selection of regions for these studies the State size and weight restrictions, the geographic features, and the predominance of certain types of vehicles favored for their adaptability to commerce or terrain of the region should be considered.

The vehicle weight comparison tables 11–17 present a reasonable nationwide picture of the relationship between recorded empty and registered gross weights of different vehicle types. These comparisons demonstrate clearly that it would not be practicable to try to develop a set of weight relationships that would permit a point, or even a narrow band, of weight conversion to be used for any purpose. Conditions in individual States may be such that modifications or adaptations of the data shown in tables 11–17 may be required before they can be applied. However, the data provide a useful tool that can serve as a guide, or reference point, for local conversion problems. The local situation would have to dictate any adjustment factors necessary to make the data in these tables applicable to the problems being considered.

able 2.—Trucks and combinations grouped by number of axles and by registered gross vehicle weights, from California interstate proration records [1]

Registered gross vehicle weight	Single-unit trucks				Combinations consisting of—											
					Tractor and semitrailer						Truck and full trailer				Tractor, semitrailer and full trailer	
	2-axles		3-axles		3-axles (2–S1)		4-axles (2–S2)		5-axles (3–S2)		3-axles (2–1)		5-axles (3–2)		5-axles (2–S1–2)	
Pounds	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*	*No.*	*Pct.*
0–3,999																
4,000–4,999																
5,000–5,999	8	1.3														
6,000–7,999	70	11.2														
8,000–9,999	40	6.4														
10,000–11,999	21	3.3					1	0.1								
12,000–13,999	15	2.4					1	0.1			1	7.7				
14,000–15,999	22	3.5														
16,000–17,999	24	3.8														
18,000–19,999	96	15.2														
20,000–21,999	74	11.7	1	2.8	2	0.1										
22,000–23,999	112	17.8			1	0.1	3	0.4			4	30.8				
24,000–25,999	85	13.5	3	8.3	9	0.7	8	1.2			4	30.8				
26,000–27,999	43	6.8	1	2.8	4	0.3					1	7.7				
28,000–29,999	8	1.3	1	2.8	3	0.2	11	1.6			2	15.3				
30,000–31,999	7	1.1			24	1.8	9	1.3					1	0.2		
32,000–35,999	3	0.5			15	1.1	2	0.3								
36,000–39,999			11	30.6	140	10.4	3	0.4	3	0.1	1	7.7				
40,000–44,999	1	0.2	17	47.2	818	60.5	8	1.2	5	0.2						
45,000–49,999			2	5.5	226	16.7	64	9.3	1				2	0.4		
50,000–54,999					81	6.0	45	6.6	8	0.3			2	0.4		
55,000–59,999					22	1.7	310	45.3	16	0.5			3	0.6		
60,000–64,999					2	0.1	207	30.2	89	3.0			2	0.4	11	2.1
65,000–69,999					1	0.1	10	1.5	289	9.6			13	2.5	1	0.2
70,000–74,999					2	0.1	2	0.3	2,160	71.6			86	16.7	101	19.5
75,000–79,999					2	0.1	1	0.2	445	14.7			406	78.8	405	78.2
80,000 and over																
TOTAL	629	100.0	36	100.0	1,352	100.0	685	100.0	3,016	100.0	13	100.0	515	100.0	518	100.0

[1] The portion of the table boxed by heavy lines represents 90 percent or more of the vehicles in each vehicle type.

'able 3.—Comparison of relative numbers of motor vehicles observed in the 1957 and 1961 loadometer studies by gross vehicle weight groups

Registered gross vehicle weight	Single-unit trucks				Vehicle combinations							
	2-axle				3-axle (2–S1)				4-axle (2–S2)			
	1957		1961		1957		1961		1957		1961	
Pounds	*Pct.*	*Cumulated Pct.*[1]	*Pct.*	*Cumulated Pct.*[1]	*Pct.*	*Cumulated Pct.*[1]	*Pct.*	*Cumulated Pct.*[1]	*Pct.*	*Cumulated Pct.*[1]	*Pct.*	*Cumulated Pct.*[1]
Under [2] 5,000	36.1	100.0	32.8	100.0								
Under [2] 18,000					(3)	(3)	0.2	100.0				
Under [2] 24,000									0.2	100.0	0.6	100.0
5,000–5,999	19.9	63.9	13.1	67.2								
6,000–7,999	9.0	44.0	15.8	54.1								
8,000–9,999	4.7	35.0	8.0	38.3								
10,000–11,999	3.9	30.3	5.4	30.3								
12,000–13,999	3.3	26.4	4.3	24.9								
14,000–15,999	3.2	23.1	1.6	20.6								
16,000–17,999	5.1	19.9	2.2	19.0								
18,000–19,999	3.3	14.8	2.6	16.8	1.9	100.0	0.3	99.8				
20,000–21,999	4.3	11.5	2.9	14.2	1.4	98.1	0.3	99.5				
22,000–23,999	2.2	7.2	2.4	11.3	1.7	96.7	0.3	99.2				
24,000–25,999	3.4	5.0	3.9	8.9	4.3	95.0	1.3	98.9	0.5	99.8	0.2	99.4
26,000–27,999	0.8	1.6	1.7	5.0	2.3	90.7	0.4	97.6	0.3	99.3	0.1	99.2
28,000–29,999	0.2	0.8	0.6	3.3	3.3	88.4	1.4	97.2	0.1	99.0	0.1	99.1
30,000–31,999	0.1	0.6	0.9	2.7	6.9	85.1	2.8	95.8	0.5	98.9	0.1	99.0
32,000–35,999	(3)	(3)	0.9	1.8	18.2	78.2	6.8	93.0	0.6	98.4	0.3	98.9
36,000–39,999	0.1	0.2	0.4	0.9	15.7	60.0	15.3	86.2	1.3	97.8	0.6	98.6
40,000–44,999	0.1	0.1	0.2	0.5	33.6	44.3	41.2	70.9	3.4	96.5	2.3	98.0
45,000–49,999	(3)	(3)	0.1	0.3	4.0	10.7	9.4	29.7	4.6	93.1	2.1	95.7
50,000–54,999	(3)	(3)	(3)	(3)	5.6	6.7	8.1	20.3	24.4	88.5	6.8	93.6
55,000–59,999	(3)	(3)	0.1	0.2	0.7	1.1	3.0	12.2	47.6	64.1	39.2	86.8
60,000–64,999					0.4	0.4	8.6	9.2	14.2	16.5	40.5	47.6
65,000–69,999									2.1	2.3	6.2	7.1
60,000 and over [2]	(3)	(3)	0.1	0.1								
65,000 and over [2]					(3)	(3)	0.6	0.6				
70,000 and over [2]									0.2	0.2	0.9	0.9
TOTAL	100.0		100.0		100.0		100.0		100.0		100.0	

[1] Percentages in this column are an inverse cumulation of the percentages in the preceding column.
[2] Open-end weight classes are shown for each visual vehicle type at the lower end and upper end of the weight classification scale. Each open-end class applies to a specific visual vehicle type.
[3] Less than 0.1 percent.

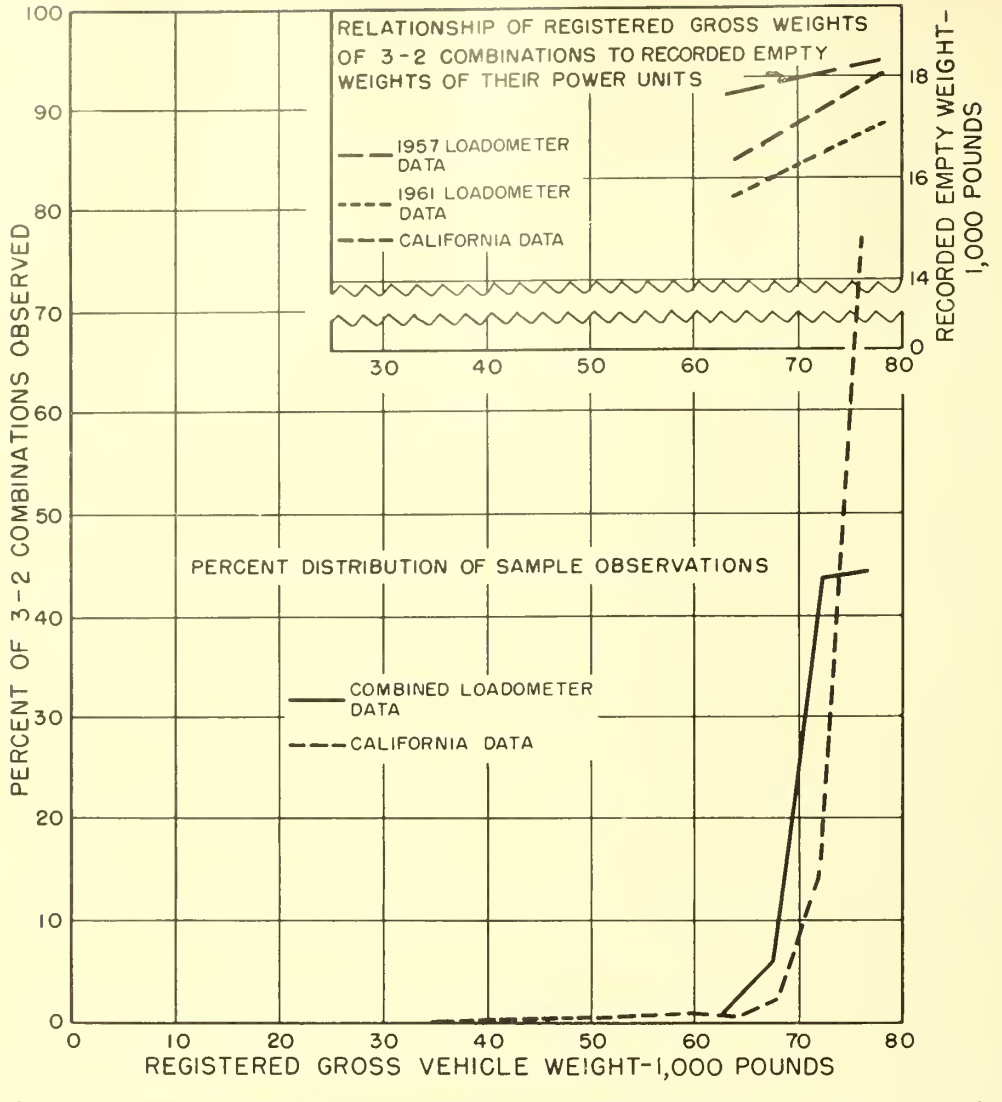
RELATIONSHIP OF REGISTERED GROSS WEIGHTS OF 3-2 COMBINATIONS TO RECORDED EMPTY WEIGHTS OF THEIR POWER UNITS
1957 LOADOMETER DATA
1961 LOADOMETER DATA
CALIFORNIA DATA
RECORDED EMPTY WEIGHT-1,000 POUNDS
PERCENT OF 3-2 COMBINATIONS OBSERVED
PERCENT DISTRIBUTION OF SAMPLE OBSERVATIONS
COMBINED LOADOMETER DATA
CALIFORNIA DATA
REGISTERED GROSS VEHICLE WEIGHT-1,000 POUNDS

Figure 8.—Empty to gross weight relationships and relative distribution of 5-axle, truck-full trailer combinations (3-2).

Table 4.—Comparison of relative numbers of motor vehicles observed in the 1957 and 1961 loadometer studies by recorded empty weights of power units

Recorded empty weight of power unit	Single-unit trucks				Vehicle combinations							
	2-axle				3-axle (2-S1)				4-axle (2-S2)			
	1957		1961		1957		1961		1957		1961	
Pounds	*Pct.*	*Cumulated Pct.*[1]	*Pct.*	*Cumulated Pct.*[1]	*Pct.*	*Cumulated Pct.*[1]	*Pct.*	*Cumulated Pct.*[1]	*Pct.*	*Cumulated Pct.*[1]	*Pct.*	*Cumulated Pct.*[1]
Under [2] 3,000	1.8	100.0	1.5	100.0								
Under [2] 5,000					4.0	100.0	0.5	100.0	0.1	100.0	0.3	100.0
3,000-3,999	42.4	98.2	39.4	98.5								
4,000-4,999	22.1	55.8	24.0	59.1								
5,000-5,999	7.8	33.7	9.5	35.1	9.1	96.0	2.6	99.5	0.6	99.9	0.8	99.7
6,000-6,999	7.8	25.9	7.2	25.6	22.0	86.9	7.3	96.9	3.7	99.3	2.8	98.9
7,000-7,999	7.3	18.1	3.6	18.4	23.4	64.9	16.7	89.6	4.7	95.6	2.6	96.1
8,000-8,999	5.1	10.8	4.1	14.8	18.4	41.5	24.3	72.9	9.7	90.9	8.5	93.5
9,000-9,999	2.7	5.7	3.5	10.7	14.3	23.1	18.3	48.6	23.7	81.2	15.0	85.0
10,000-10,999	1.4	3.0	2.6	7.2	5.2	8.8	15.8	30.3	26.0	57.5	23.1	70.0
11,000-11,999	0.6	1.6	1.8	4.6	3.6	3.6	6.8	14.5	12.4	31.5	20.4	46.9
12,000-12,999	0.4	1.0	1.1	2.8					12.4	19.1	18.2	26.5
13,000-13,999									4.5	6.7	5.7	8.3
12,000 and over [2]					(3)	(3)	7.7	7.7				
13,000 and over [2]	0.6	0.6	1.7	1.7								
14,000 and over [2]									2.2	2.2	2.6	2.6
TOTAL	100.0		100.0		100.0		100.0		100.0		100.0	

[1] Percentages in this column are an inverse cumulation of the percentages in the preceding column.
[2] Open-end weight classes are shown for each visual vehicle type at the lower end and upper end of the weight classification scale. Each open-end class applies to only one visual vehicle type.
[3] Less than 0.1 percent.

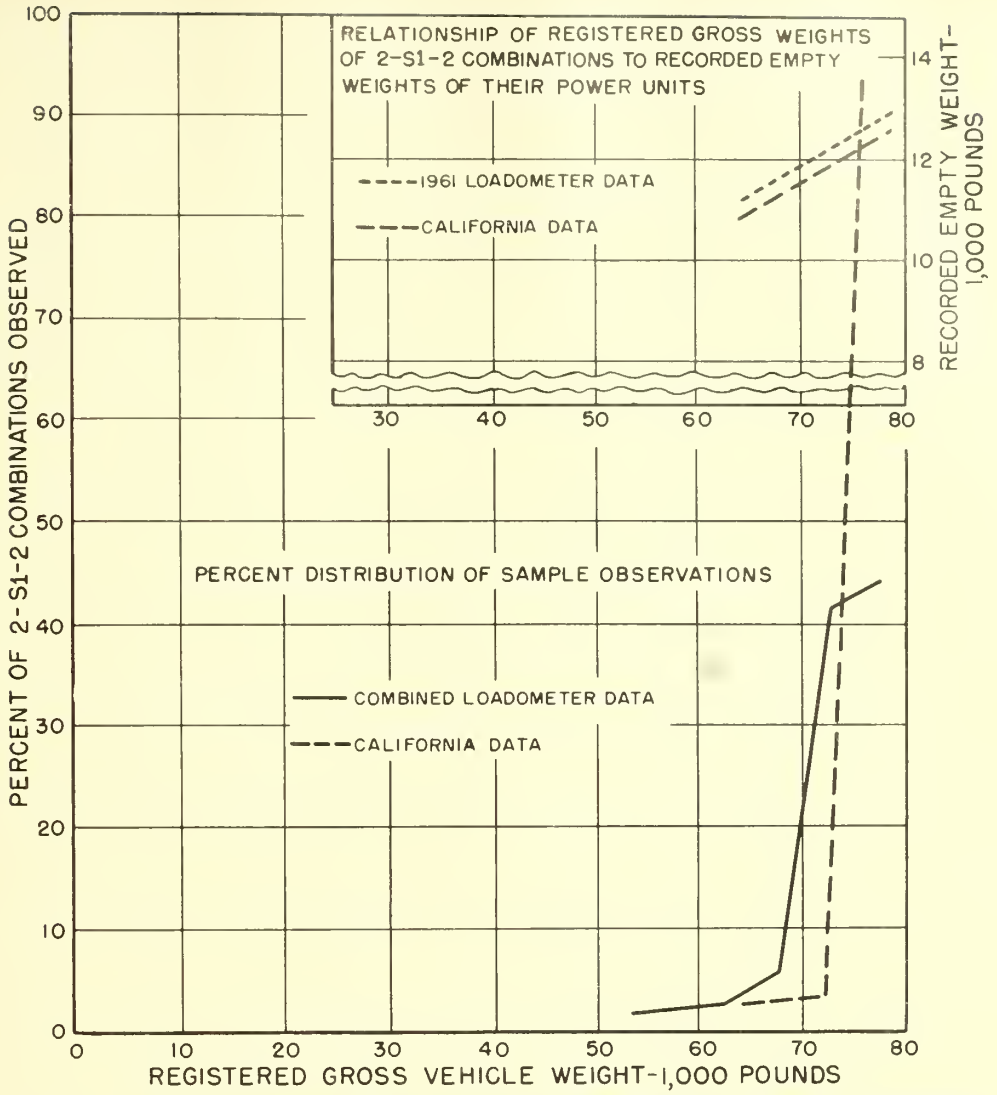
RELATIONSHIP OF REGISTERED GROSS WEIGHTS OF 2-S1-2 COMBINATIONS TO RECORDED EMPTY WEIGHTS OF THEIR POWER UNITS
1961 LOADOMETER DATA
CALIFORNIA DATA
RECORDED EMPTY WEIGHT-1,000 POUNDS
PERCENT DISTRIBUTION OF SAMPLE OBSERVATIONS
COMBINED LOADOMETER DATA
CALIFORNIA DATA
PERCENT OF 2-S1-2 COMBINATIONS OBSERVED
REGISTERED GROSS VEHICLE WEIGHT-1,000 POUNDS

Figure 9.—Empty to gross weight relationships and relative distribution of 5-axle, tractor-semitrailer, full trailer combinations (2-S1-2).

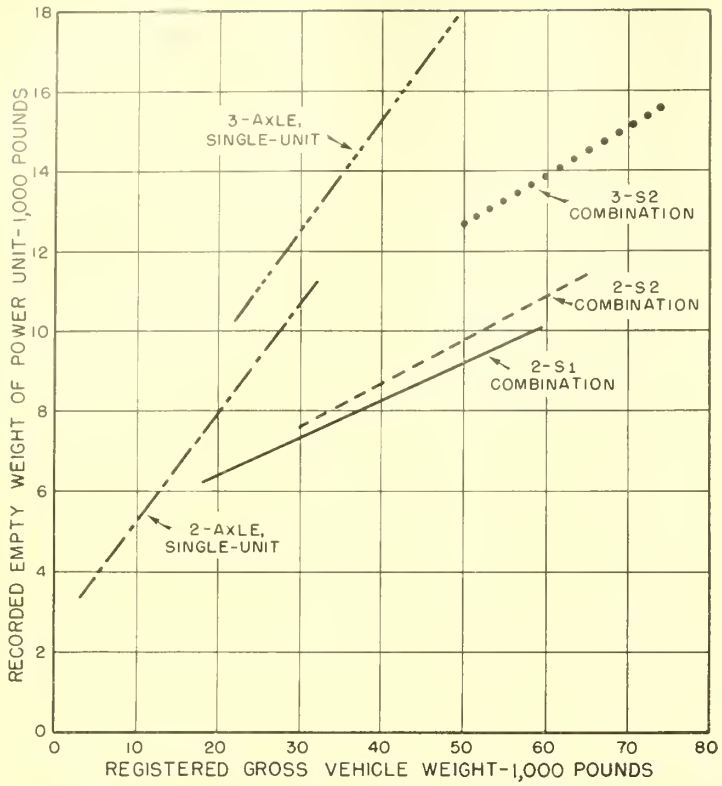

Figure 10.—Relationship of the recorded empty weights of the power units to the registered gross weights of the vehicles based on combined 1957 and 1961 loadometer data.

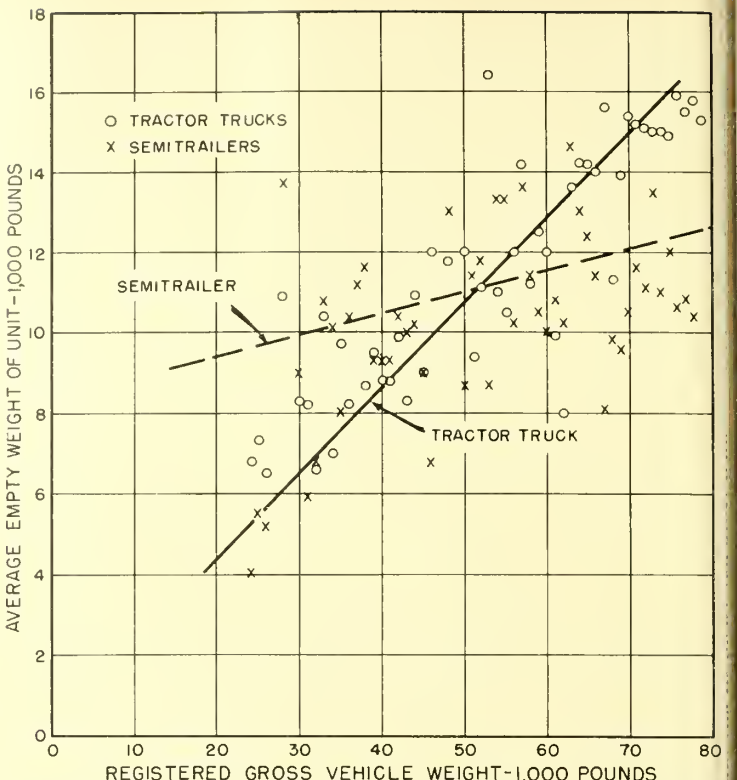

Figure 11.—Scattergram of average empty weight of tractor trucks and of semitrailers by registered gross combination weight, and lines of best fit (California data).

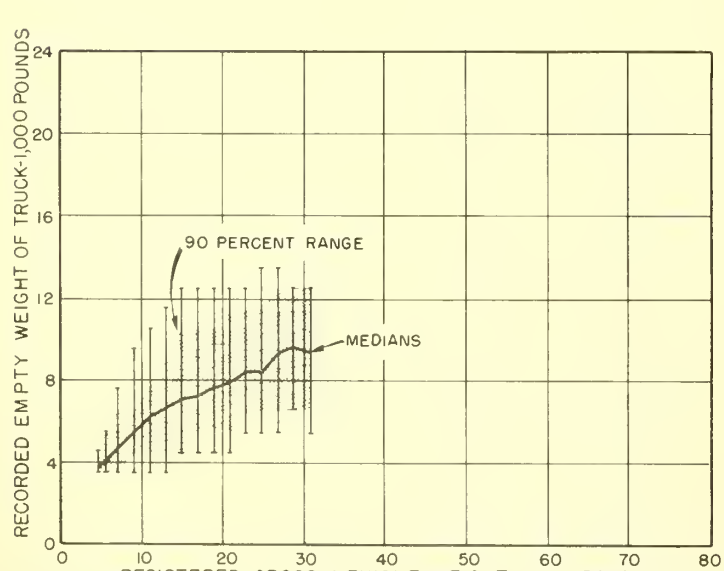

Figure 12.—Range of recorded empty weights of 2-axle trucks registered by gross vehicle weights, based on the combined 1957 and 1961 loadometer data.

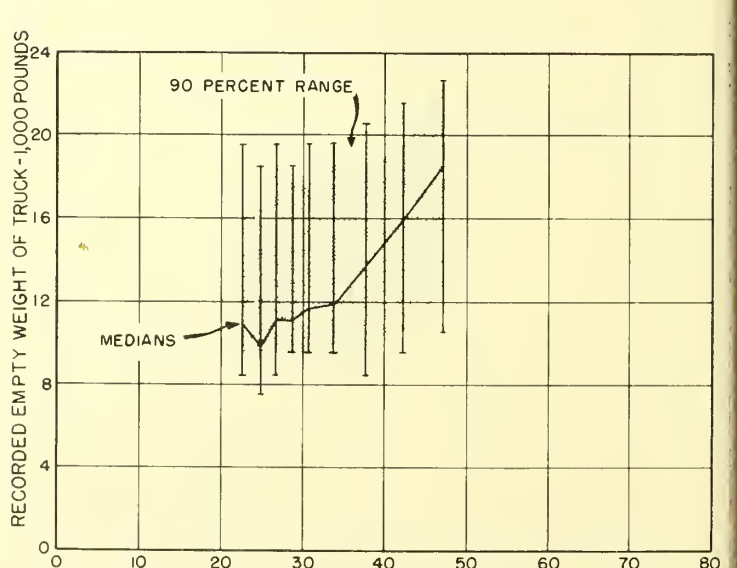

Figure 13.—Range of recorded empty weights of 3-axle trucks registered by gross vehicle weights, based on the combined 1957 and 1961 loadometer data.

Table 5.—Comparison of number and percent of 2-axle, single-unit trucks by recorded empty weights and by registered gross vehicle weights, 1957 loadometer data [1]

Recorded empty weight of truck (pounds)	Registered gross vehicle weight (pounds)																						Total	
	4,000–4,999	5,000–5,999	6,000–7,999	8,000–9,999	10,000–11,999	12,000–13,999	14,000–15,999	16,000–17,999	18,000–19,999	20,000–21,999	22,000–23,999	24,000–25,999	26,000–27,999	28,000–29,999	30,000–31,999	32,000–35,999	36,000–39,999	40,000–44,999	45,000–49,999	50,000–54,999	55,000–59,999	60,000 and over	Number	Percent of total
0–2,999:																								
Number	1,614	621	66	1	2	3	1				2	1											2,311	1.8
Percent	69.9	26.9	2.9	(2)	0.1	0.1	(2)				0.1	(2)												
3,000–3,999:																								
Number	34,176	15,530	4,804	955	123	22	11	6	4	1													55,632	42.4
Percent	61.5	27.9	8.7	1.7	0.2	(2)	(2)	(2)	(2)	(2)														
4,000–4,999:																								
Number	11,618	7,968	4,948	2,079	1,223	459	220	273	145	79	10		1		2		1	1		1			29,028	22.1
Percent	40.0	27.5	17.0	7.2	4.2	1.6	0.8	0.9	0.5	0.3	(2)		(2)		(2)		(2)	(2)		(2)				
5,000–5,999:																								
Number		1,979	1,738	1,793	1,388	945	689	893	335	208	79	109	27	5	10		3			2			10,203	7.8
Percent		19.4	17.0	17.6	13.6	9.3	6.7	8.8	3.3	2.0	0.8	1.1	0.3	(2)	0.1		(2)			(2)				
6,000–6,999:																								
Number			300	1,058	1,438	1,312	1,103	1,854	976	1,176	342	501	120	9	43	6	8	15	6	2		2	10,281	7.8
Percent			2.9	10.4	14.0	12.8	10.7	18.0	9.5	11.4	3.3	4.9	1.2	0.1	0.4	0.1	0.1	0.1	0.1	(2)		(2)		
7,000–7,999:																								
Number			10	257	758	730	967	1,832	1,107	1,672	646	1,369	160	22	80	5	12	11	13	2		1	9,654	7.3
Percent			0.1	2.7	7.8	7.6	10.0	19.0	11.5	17.3	6.7	14.2	1.7	0.2	0.8	0.1	0.1	0.1	0.1	(2)		(2)		
8,000–8,999:																								
Number				22	184	533	511	1,011	886	1,181	802	1,201	186	31	103	6	13	13	6	6		5	6,700	5.1
Percent				0.3	2.7	8.0	7.6	15.1	13.2	17.6	12.0	17.9	2.8	0.5	1.5	0.1	0.2	0.2	0.1	0.1		0.1		
9,000–9,999:																								
Number				6	23	245	321	453	474	698	414	539	177	60	75	1	5	5	1	1	1	15	3,514	2.7
Percent				0.2	0.7	7.0	9.2	12.9	13.5	19.9	11.8	15.4	5.0	1.7	2.1	(2)	0.1	0.1	(2)	(2)	(2)	0.4		
10,000–10,999:																								
Number					7	51	246	214	154	288	291	311	136	37	38	2	13	5	3	3	1	23	1,823	1.4
Percent					0.4	2.8	13.5	11.7	8.4	15.8	15.9	17.1	7.5	2.0	2.1	0.1	0.7	0.3	0.2	0.2	(2)	1.3		
11,000–11,999:																								
Number						8	63	117	62	94	132	187	81	20	34	5	3	3	3			6	818	0.6
Percent						1.0	7.7	14.3	7.6	11.5	16.1	22.9	9.9	2.4	4.1	0.6	0.4	0.4	0.4			0.7		
12,000–12,999:																								
Number						4	12	51	58	88	71	91	79	35	23	11	6	1	4				534	0.4
Percent						0.7	2.2	9.6	10.9	16.5	13.3	17.0	14.8	6.6	4.3	2.1	1.1	0.2	0.7					
13,000 and over:																								
Number						1	2	23	79	109	76	201	87	39	61	16	19	32		3	4		752	0.6
Percent						0.1	0.3	3.1	10.5	14.5	10.1	26.7	11.6	5.2	8.1	2.1	2.5	4.3		0.4	0.5			
TOTAL:																								
Number	47,408	26,098	11,866	6,181	5,146	4,313	4,146	6,727	4,280	5,594	2,865	4,510	1,054	258	469	52	83	86	36	20	6	52	131,250	100.0
Percent	36.1	19.9	9.0	4.7	3.9	3.3	3.2	5.1	3.3	4.3	2.2	3.4	0.8	0.2	0.4	(2)	0.1	0.1	(2)	(2)	(2)	(2)		

[1] The portion of the table boxed by heavy lines represents 90 percent or more of the vehicles in each empty weight group.
[2] Less than 0.1 percent.

Table 6.—Comparison of number and percent of 2-axle, single-unit trucks by recorded empty weights and by registered gross vehicle weights, 1961 loadometer data[1]

Recorded empty weight of truck (pounds)	Registered gross vehicle weight (pounds)																					Total number	Percent of total
	4,000–4,999	5,000–5,999	6,000–7,999	8,000–9,999	10,000–11,999	12,000–13,999	14,000–15,999	16,000–17,999	18,000–19,999	20,000–21,999	22,000–23,999	24,000–25,999	26,000–27,999	28,000–29,999	30,000–31,999	32,000–35,999	36,000–39,999	40,000–44,999	45,000–49,999	50,000–54,999	55,000–59,999	60,000 and over		
0–2,999:																							
Number	64	18	5					1														88	1.5
Percent	72.7	20.5	5.7					1.1															
3,000–3,999:																							
Number	1,348	445	335	73	9	2	2	2	1			1									1	2,249	39.4
Percent	60.0	19.8	16.2	3.3	0.4	0.1	0.1	0.1	(2)			(2)									(2)		
4,000–4,999:																							
Number	459	228	376	159	86	34	9	1	4	9	2	4									1	1,372	24.0
Percent	33.4	16.6	27.4	11.6	6.3	2.5	0.6	0.1	0.3	0.7	0.1	0.3									0.1		
5,000–5,999:																							
Number		57	132	137	97	71	8	17	6	3	5	4		1		3						541	9.5
Percent		10.5	24.4	25.3	17.9	13.1	1.5	3.2	1.1	0.6	0.9	0.7		0.2		0.6							
6,000–6,999:																							
Number			23	82	101	99	20	27	19	11	8	12	3	3		1						409	7.2
Percent			5.6	20.1	24.7	24.2	4.9	6.6	4.7	2.7	2.0	2.9	0.7	0.7		0.2							
7,000–7,999:																							
Number				2	10	17	12	28	28	29	24	44	8		3	1	1				1	208	3.6
Percent				1.0	4.8	8.2	5.8	13.5	13.5	13.9	11.5	21.1	3.8		1.4	0.5	0.5				0.5		
8,000–8,999:																							
Number				2	2	13	20	19	36	35	33	54	8	6	5	2	1					236	4.1
Percent				0.8	0.8	5.5	8.5	8.1	15.3	14.8	14.0	22.9	3.4	2.6	2.1	0.8	0.4						
9,000–9,999:																							
Number					2	6	12	13	30	26	30	36	23	5	8	4	1	1				197	3.5
Percent					1.0	3.1	6.1	6.6	15.2	13.2	15.2	18.3	11.7	2.5	4.1	2.0	0.5	0.5					
10,000–10,999:																							
Number					1	2	4	7	12	25	15	29	28	4	6	7	3	1	1		1	146	2.6
Percent					0.7	1.4	2.7	4.8	8.2	17.1	10.3	19.9	19.2	2.7	4.1	4.8	2.0	0.7	0.7		0.7		
11,000–11,999:																							
Number				1	2	2	1	5	11	14	7	18	12	7	8	7	2	3			1	101	1.8
Percent				1.0	2.0	2.0	1.0	4.9	10.9	13.9	6.9	17.8	11.9	6.9	7.9	6.9	2.0	3.0			1.0		
12,000–12,999:																							
Number						1	1	5	1	9	3	7	12	7	6	7	2		2			63	1.1
Percent						1.6	1.6	7.9	1.6	14.3	4.8	11.1	19.0	11.1	9.5	11.1	3.2		3.2				
13,000 and over:																							
Number							1	3	3	6	8	13	5	3	15	19	10	6	2	1	2	97	1.7
Percent							1.0	3.1	3.1	6.2	8.2	13.4	5.1	3.1	15.5	19.6	10.3	6.2	2.1	1.0	2.1		
TOTAL:																							
Number	1,871	748	901	456	310	247	90	128	151	167	135	222	99	36	51	51	20	11	5	1	3	4	5,707	100
Percent	32.8	13.1	15.8	8.0	5.4	4.3	1.6	2.2	2.6	2.9	2.4	3.9	1.7	0.6	0.9	0.9	0.4	0.2	0.1	0.0	0.1	0.1	

[1] The portion of the table boxed by heavy lines represents 90 percent or more of the vehicles in each empty weight group.
[2] Less than 0.1 percent.

Table 7.—Comparison of number and percent of 3-axle, tractor-semitrailer combinations (2–S1) by tractor recorded empty weights and by registered gross vehicle weights, 1957 loadometer data [1]

Recorded empty weight of tractor (pounds)	Registered gross combination weight (pounds)															Total number	Percent of total
	0–17,999	18,000–19,999	20,000–21,999	22,000–23,999	24,000–25,999	26,000–27,999	28,000–29,999	30,000–31,999	32,000–35,999	36,000–39,999	40,000–44,999	45,000–49,999	50,000–54,999	55,000–59,999	60,000–64,999		
0–4,999:																	
Number		28	4	6	18	12	10	17	20	36	58	3	1		1	214	4.0
Percent		13.1	1.9	2.8	8.4	5.6	4.7	7.9	9.3	16.8	27.1	1.4	0.5		0.5		
5,000–5,999:																	
Number		25	15	39	48	16	32	40	88	64	97	12	10	1	2	489	9.1
Percent		5.1	3.1	8.0	9.8	3.3	6.5	8.2	18.0	13.1	19.8	2.5	2.0	0.2	0.4		
6,000–6,999:																	
Number		23	14	13	79	32	54	78	347	219	282	24	17			1,182	22.0
Percent		1.9	1.2	1.1	6.7	2.7	4.6	6.6	29.4	18.5	23.9	2.0	1.4				
7,000–7,999:																	
Number		17	28	18	34	20	25	146	315	263	336	24	28	1	2	1,257	23.4
Percent		1.4	2.2	1.4	2.7	1.6	2.0	11.6	25.1	20.9	26.7	1.9	2.2	0.1	0.2		
8,000–8,999:																	
Number		8	8	8	36	28	33	47	93	146	457	60	51	7	3	985	18.4
Percent		0.8	0.8	0.8	3.7	2.8	3.4	4.8	9.4	14.8	46.4	6.1	5.2	0.7	0.3		
9,000–9,999:																	
Number		2	5	4	8	10	11	26	70	68	360	67	120	14	2	767	14.3
Percent		0.3	0.7	0.5	1.1	1.3	1.4	3.4	9.1	8.9	46.9	8.7	15.6	1.8	0.3		
10,000–10,999:																	
Number				1	3	3	5	6	29	30	129	16	40	8	7	277	5.2
Percent				0.4	1.1	1.1	1.8	2.1	10.5	10.8	46.6	5.8	14.4	2.9	2.5		
11,000–11,999:																	
Number				1	3	2	4	8	15	19	86	8	34	7	7	194	3.6
Percent				0.5	1.6	1.0	2.1	4.1	7.8	9.8	44.3	4.1	17.5	3.6	3.6		
TOTAL:																	
Number		103	74	90	229	123	174	368	977	845	1,805	214	301	38	24	5,365	100.0
Percent		1.9	1.4	1.7	4.3	2.3	3.3	6.9	18.2	15.7	33.6	4.0	5.6	0.7	0.4		

[1] The portion of the table boxed by heavy lines represents 90 percent or more of the vehicles in each empty weight group.

Table 8.—Comparison of number and percent of 3-axle, tractor-semitrailer combinations (2–S1) by tractor recorded empty weights and by registered gross vehicle weights, 1961 loadometer data [1]

Recorded empty weight of tractor (pounds)	Registered gross combination weight (pounds)																Total number	Percent of total
	0–17,999	18,000–19,999	20,000–21,999	22,000–23,999	24,000–25,999	26,000–27,999	28,000–29,999	30,000–31,999	32,000–35,999	36,000–39,999	40,000–44,999	45,000–49,999	50,000–54,000	55,000–59,999	60,000–64,999	65,000 and over		
0–4,999:																		
Number		2	1								2						5	0.5
Percent		40.0	20.0								40.0							
5,000–5,999:																		
Number							1	1	8	5	7	1	1				24	2.6
Percent							4.2	4.2	33.3	20.8	29.1	4.2	4.2					
6,000–6,999:																		
Number		1		1	1	2	1	5	14	10	32	1					68	7.3
Percent		1.5		1.5	1.5	2.9	1.5	7.3	20.6	14.7	47.0	1.5						
7,000–7,999:																		
Number				1	3	1	6	5	14	47	68	3	3	4			155	16.7
Percent				0.7	1.9	0.7	3.9	3.2	9.0	30.3	43.9	1.9	1.9	2.6				
8,000–8,999:																		
Number	2		1	1	4	1	3	7	14	37	115	17	17	3	3	1	226	24.3
Percent	0.9		0.4	0.5	1.8	0.5	1.3	3.1	6.2	16.4	50.9	7.5	7.5	1.3	1.3	0.4		
9,000–9,999:																		
Number			1		3			5	7	24	77	21	14	5	13		170	18.3
Percent			0.6		1.8			3.0	4.1	14.1	45.3	12.4	8.2	2.9	7.6			
10,000–10,999:																		
Number					1		1	2	2	10	44	25	26	6	28	2	147	15.8
Percent					0.7		0.7	1.4	1.4	6.8	29.9	17.0	17.7	4.1	19.0	1.3		
11,000–11,999:																		
Number									4	3	22	5	6	4	17	2	63	6.8
Percent									6.4	4.8	34.9	7.9	9.5	6.3	27.0	3.2		
12,000 and over:																		
Number							1	1		6	16	14	8	6	19	1	72	7.7
Percent							1.4	1.4		8.3	22.2	19.5	11.1	8.3	26.4	1.4		
TOTAL:																		
Number	2	3	3	3	12	4	13	26	63	142	383	87	75	28	80	6	930	100.0
Percent	0.2	0.3	0.3	0.3	1.3	0.4	1.4	2.8	6.8	15.3	41.2	9.4	8.1	3.0	8.6	0.6		

[1] The portion of the table boxed by heavy lines represents 90 percent or more of the vehicles in each empty weight group.

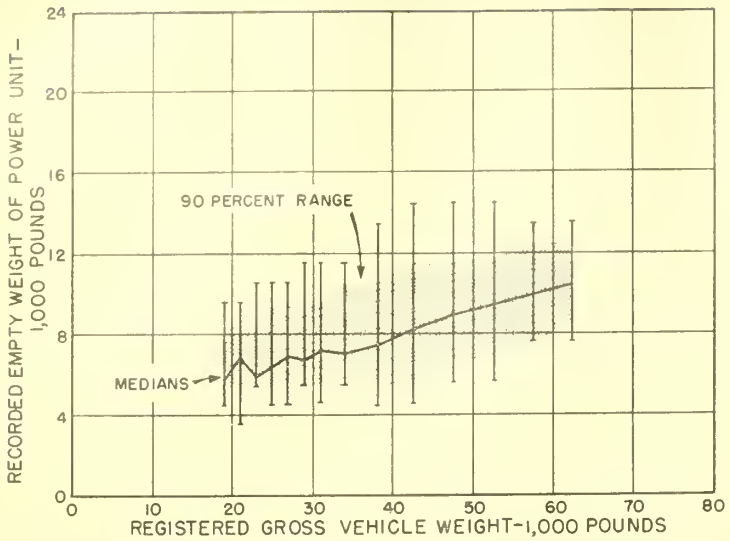

Figure 14.—Range of recorded tractor empty weights of 3-axle, tractor-semitrailer combinations (2-S1) registered by gross vehicle weights, based on the combined 1957 and 1961 loadometer data.

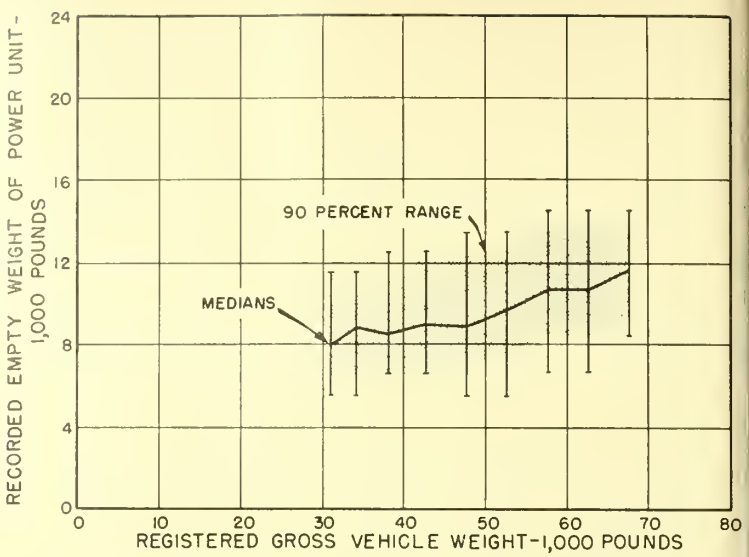

Figure 15.—Range of recorded tractor empty weights of 4-axle tractor-semitrailer combinations (2-S2) registered by gross vehicle weights, based on the combined 1957 and 1961 loadometer data.

Table 9.—Comparison of number and percent of 4-axle, tractor-semitrailer combinations (2-S2) by tractor recorded empty weights and by registered gross vehicle weights, 1957 loadometer data [1]

Recorded empty weight of tractor (pounds)	Registered gross combination weight (pounds)													Total number	Percent of total
	0-23,999	24,000-25,999	26,000-27,999	28,000-29,999	30,000-31,999	32,000-35,999	36,000-39,999	40,000-44,999	45,000-49,999	50,000-54,999	55,000-59,999	60,000-64,999	65,000-69,999	70,000 and over	
0-4,999:															
Number	2							3	2						7
Percent	28.6							42.8	28.6						0.1
5,000-5,999:															
Number	1	3	7	2	2	1	1	2	7	13	2	1			42
Percent	2.4	7.1	16.6	4.8	4.8	2.4	2.4	4.8	16.6	30.9	4.8	2.4			0.6
6,000-6,999:															
Number	6	16	3	2	6	7	23	31	71	50	38	6			259
Percent	2.3	6.2	1.1	0.8	2.3	2.7	8.9	12.0	27.4	19.3	14.7	2.3			3.7
7,000-7,999:															
Number	1	3	2	3	10	8	11	43	32	118	70	27		1	329
Percent	0.3	0.9	0.6	0.9	3.0	2.4	3.4	13.1	9.7	35.9	21.3	8.2		0.3	4.7
8,000-8,999:															
Number	2	1	4	1	1	8	18	56	53	264	182	95			685
Percent	0.3	0.1	0.6	0.1	0.1	1.2	2.6	8.2	7.7	38.6	26.6	13.9			9.7
9,000-9,999:															
Number	4	3	2	1	11	12	27	54	79	546	648	279	4	2	1,672
Percent	0.2	0.2	0.1	0.1	0.7	0.7	1.6	3.2	4.7	32.7	38.8	16.7	0.2	0.1	23.7
10,000-10,999:															
Number	1	2	1	1	2	4	5	28	42	417	985	310	35	2	1,835
Percent	0.1	0.1	0.1	0.1	0.1	0.2	0.3	1.5	2.3	22.7	53.6	16.9	1.9	0.1	26.0
11,000-11,999:															
Number			1		4	1	3	11	8	190	505	107	40	2	872
Percent			0.1		0.5	0.1	0.3	1.3	0.9	21.8	57.9	12.3	4.6	0.2	12.4
12,000-12,999:															
Number	1	1					1	7	13	83	650	91	21	5	873
Percent	0.1	0.1					0.1	0.8	1.5	9.5	74.5	10.4	2.4	0.6	12.4
13,000-13,999:															
Number		3					1	3	7	18	206	41	39		318
Percent		0.9					0.3	0.9	2.2	5.7	64.8	12.9	12.3		4.5
14,000 and over:															
Number								1	8	21	64	44	9	5	152
Percent								0.7	5.3	13.8	42.1	28.9	5.9	3.3	2.2
TOTAL:															
Number	18	32	20	10	36	41	90	239	322	1,720	3,350	1,001	148	17	7,044
Percent	0.2	0.5	0.3	0.1	0.5	0.6	1.3	3.4	4.6	24.4	47.6	14.2	2.1	0.2	100.0

[1] The portion of the table boxed by heavy lines represents 80 percent or more of the vehicles in each empty weight group.

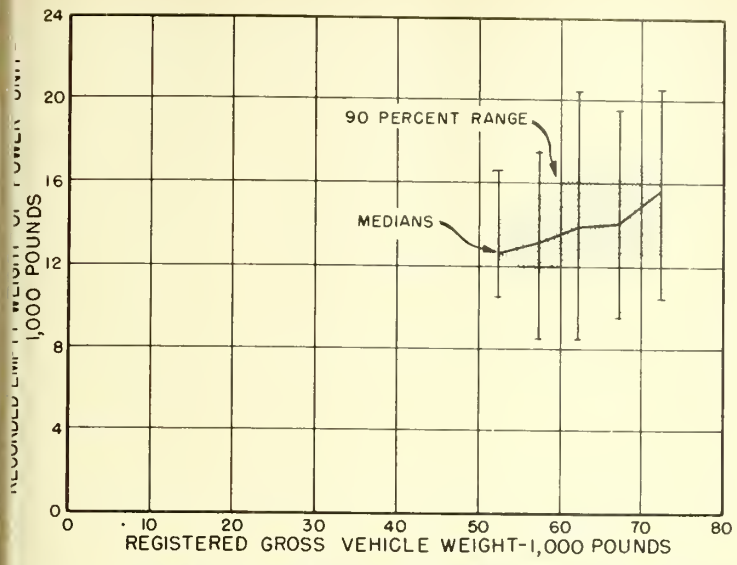

Figure 16.—Range of recorded tractor empty weights of 5-axle, tractor-semitrailer combinations (3-S2) registered by gross vehicle weights, based on the combined 1957 and 1961 loadometer data.

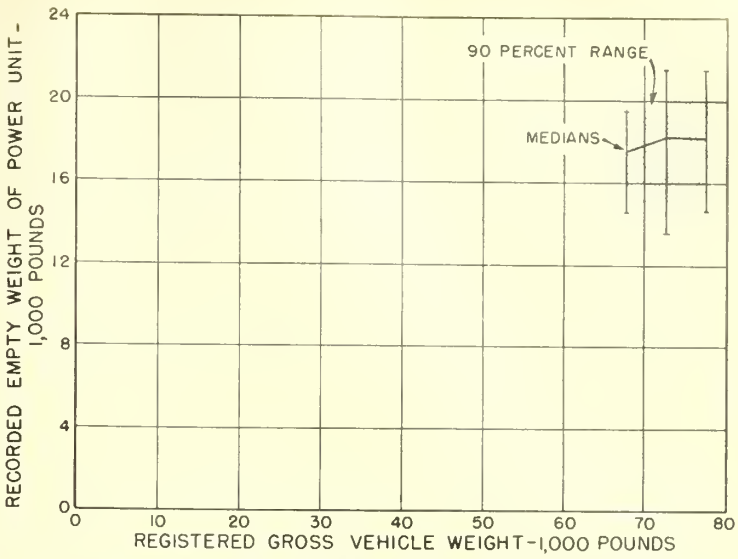

Figure 17.—Range of recorded truck empty weights of 5-axle, truck full-trailer combinations (3-2) registered by gross vehicle weights, based on the combined 1957 and 1961 loadometer data.

Table 10.—Comparison of number and percent of 4-axle, tractor-semitrailer combinations (2-S2) by tractor recorded empty weights and by registered gross vehicle weights, 1961 loadometer data[1]

Recorded empty weight of tractor (pounds)	Registered gross combination weight (pounds)														Total number	Percent of total
	0–23,999	24,000–25,999	26,000–27,999	28,000–29,999	30,000–31,999	32,000–35,999	36,000–39,999	40,000–44,999	45,000–49,999	50,000–54,999	55,000–59,999	60,000–64,999	65,000–69,999	70,000 and over		
0–4,999:																
Number		1							1	2		1		1	6	0.3
Percent		16.6							16.7	33.3		16.7		16.7		
5,000–5,999:																
Number	2	1							1	4	5	1			14	0.8
Percent	14.3	7.1							7.2	28.6	35.7	7.1				
6,000–6,999:																
Number	2	1			1			1	5	14	22	4			50	2.8
Percent	4.0	2.0			2.0			2.0	10.0	28.0	44.0	8.0				
7,000–7,999:																
Number							4		3	9	21	11			48	2.6
Percent							8.3		6.3	18.8	43.7	22.9				
8,000–8,999:																
Number	1					1	4	5	5	27	87	22	1	1	154	8.5
Percent	0.6					0.6	2.6	3.3	3.3	17.5	56.5	14.3	0.7	0.6		
9,000–9,999:																
Number						2	3	6	8	27	136	82	9		273	15.0
Percent						0.7	1.1	2.2	2.9	9.9	49.8	30.1	3.3			
10,000–10,999:																
Number	4			1	1	2		6	5	22	139	205	31	3	419	23.1
Percent	1.0			0.2	0.2	0.5		1.4	1.2	5.3	33.2	48.9	7.4	0.7		
11,000–11,999:																
Number						1		8	2	7	147	167	37	2	371	20.4
Percent						0.3		2.2	0.5	1.9	39.6	45.0	10.0	0.5		
12,000–12,999:																
Number	2		2					5	6	9	109	169	24	5	331	18.2
Percent	0.6		0.6					1.5	1.8	2.7	32.9	51.1	7.3	1.5		
13,000–13,999:																
Number								10	3	2	25	50	10	3	103	5.7
Percent								9.7	2.9	2.0	24.3	48.5	9.7	2.9		
14,000 and over:																
Number											20	24	1	2	47	2.6
Percent											42.5	51.1	2.1	4.3		
TOTAL:																
Number	11	3	2	1	2	6	11	41	39	123	711	736	113	17	1,816	100.0
Percent	0.6	0.2	0.1	0.1	0.1	0.3	0.6	2.3	2.1	6.8	39.2	40.5	6.2	0.9		

[1] The portion of the table boxed by heavy lines represents 90 percent or more of the vehicles in each empty weight group.

Table 11.—Table for estimating the distribution of 2-axle, single-unit trucks grouped by recorded empty weights, by groups of probable registered gross vehicle weights

Recorded empty weight of truck (pounds)	Registered gross vehicle weight (pounds)																					Total number	Percent of total
	4,000–4,999	5,000–5,999	6,000–7,999	8,000–9,999	10,000–11,999	12,000–13,999	14,000–15,999	16,000–17,999	18,000–19,999	20,000–21,999	22,000–23,999	24,000–25,999	26,000–27,999	28,000–29,999	30,000–31,999	32,000–35,999	36,000–39,999	40,000–44,999	45,000–49,999	50,000–54,999	55,000–59,999	60,000 and over		
0–2,999:																							
Number	1,678	639	71	1	2	3	1	1			2	1										2,399	1.8
Percent	70.0	26.7	3.0	(1)	0.1	0.1	(1)	(1)			0.1	(1)											
3,000–3,999:																							
Number	35,524	15,975	5,169	1,028	132	24	13	8	5	1		1									1	57,881	42.3
Percent	61.4	27.6	9.0	1.8	0.2	(1)	(1)	(1)	(1)	(1)		(1)									(1)		
4,000–4,999:																							
Number	12,077	8,196	5,324	2,238	1,309	493	229	274	149	88	12	4	1		2		1	1		1		1	30,400	22.2
Percent	39.7	27.0	17.5	7.4	4.3	1.6	0.8	0.9	0.5	0.3	(1)	(1)	(1)		(1)		(1)	(1)		(1)		(1)	
5,000–5,999:																							
Number		2,036	1,870	1,930	1,485	1,016	697	910	341	211	84	113	27	6	10	3	3			2			10,744	7.8
Percent		19.0	17.4	18.0	13.8	9.5	6.5	8.5	3.2	2.0	0.8	1.1	0.2	(1)	(1)	(1)	(1)			(1)			
6,000–6,999:																							
Number			323	1,150	1,539	1,411	1,123	1,881	995	1,187	350	513	123	12	43	7	8	15	6	2		2	10,690	7.8
Percent			3.0	10.8	14.4	13.2	10.5	17.6	9.3	11.1	3.3	4.8	1.2	0.1	0.4	0.1	0.1	0.1	(1)	(1)		(1)	
7,000–7,999:																							
Number			10	259	768	747	979	1,860	1,135	1,701	670	1,413	168	22	83	6	13	11	13	2	1	1	9,862	7.2
Percent			0.1	2.6	7.8	7.6	9.9	18.9	11.5	17.3	6.8	14.3	1.7	0.2	0.9	0.1	0.1	0.1	0.1	(1)	(1)	(1)	
8,000–8,999:																							
Number				24	186	546	531	1,030	922	1,216	835	1,255	194	37	108	8	14	13	6	6		5	6,936	5.1
Percent				0.3	2.7	7.9	7.7	14.9	13.3	17.5	12.0	18.1	2.8	0.5	1.6	0.1	0.2	0.2	0.1	0.1		(1)	
9,000–9,999:																							
Number				6	25	251	333	466	504	724	444	575	200	65	83	5	6	6	1	1	1	15	3,711	2.7
Percent				0.2	0.7	6.7	9.0	12.6	13.6	19.5	12.0	15.5	5.4	1.7	2.2	0.1	0.2	0.2	(1)	(1)	(1)	0.4	
10,000–10,999:																							
Number					8	53	250	221	166	313	306	340	164	41	44	9	16	6	4	3	1	24	1,969	1.4
Percent					0.4	2.7	12.7	11.2	8.4	15.9	15.5	17.3	8.3	2.1	2.2	0.5	0.8	0.3	0.2	0.2	0.1	1.2	
11,000–11,999:																							
Number				1	2	10	64	122	73	108	139	205	93	27	42	12	5	6	3		1	6	919	0.7
Percent				0.1	0.2	1.1	7.0	13.3	7.9	11.8	15.1	22.3	10.1	2.9	4.6	1.3	0.5	0.7	0.3		0.1	0.7	
12,000–12,999:																							
Number						5	13	56	59	97	74	98	91	42	29	18	8	1	6				597	0.4
Percent						0.8	2.2	9.4	9.9	16.3	12.4	16.4	15.2	7.0	4.9	3.0	1.3	0.2	1.0				
13,000 and over:																							
Number						1	3	26	82	115	84	214	92	42	76	35	29	38	2	4	4	2	849	0.6
Percent						0.1	0.4	3.1	9.7	13.5	9.9	25.2	10.8	4.9	9.0	4.1	3.4	4.5	0.2	0.5	0.5	0.2	
TOTAL:																							
Number	49,279	26,846	12,767	6,637	5,456	4,560	4,236	6,855	4,431	5,761	3,000	4,732	1,153	294	520	103	103	97	41	21	9	56	136,957	100.0
Percent	36.0	19.6	9.3	4.9	4.0	3.3	3.1	5.0	3.2	4.2	2.2	3.5	0.8	0.2	0.4	0.1	0.1	0.1	(1)	(1)	(1)	(1)	

[1] Less than 0.1 percent.

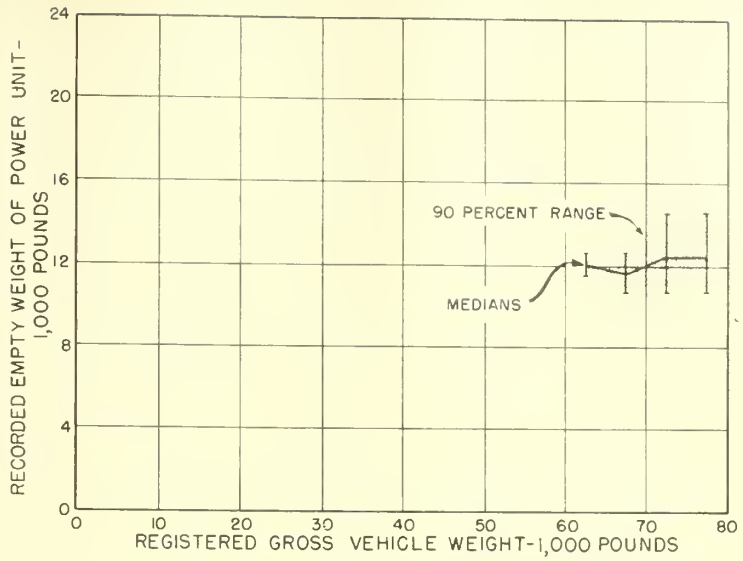

Figure 18.—Range of recorded tractor empty weights of 5-axle, tractor-semitrailer, full-trailer combinations (2–S1–2) registered by gross vehicle weights, based on the combined 1957 and 1961 loadometer data.

Table 12.—Table for estimating the distribution of 3-axle, single-unit trucks grouped by recorded empty weights, by groups of probable registered gross vehicle weights

Recorded empty weight of truck (pounds)	Registered gross vehicle weight (pounds)													Total number	Percent of total
	Under 18,000	18,000–19,999	20,000–21,999	22,000–23,999	24,000–25,999	26,000–27,999	28,000–29,999	30,000–31,999	32,000–35,999	36,000–39,999	40,000–44,999	45,000–49,999	50,000 and over		
Under 9,000:															
Number	99	33	34	29	63	42	42	58	51	54	99	9	1	614	8.3
Percent	16.1	5.4	5.5	4.7	10.3	6.8	6.8	9.5	8.3	8.8	16.1	1.5	0.2		
9,000–9,999:															
Number	21	3	11	16	52	17	32	51	93	69	42	2	2	411	5.6
Percent	5.1	0.7	2.7	3.9	12.7	4.1	7.8	12.4	22.6	16.8	10.2	0.5	0.5		
10,000–10,999:															
Number	11	4	6	9	23	41	30	36	145	137	63	4	----	509	6.9
Percent	2.1	0.8	1.2	1.8	4.5	8.0	5.9	7.1	28.5	26.9	12.4	0.8	----		
11,000–11,999:															
Number	8	1	5	19	19	20	38	28	70	133	72	7	4	424	5.7
Percent	1.9	0.2	1.2	4.5	4.5	4.7	9.0	6.6	16.5	31.4	17.0	1.6	0.9		
12,000–12,999:															
Number	7	1	2	4	11	12	15	18	63	134	140	101	21	529	7.2
Percent	1.3	0.2	0.4	0.7	2.1	2.3	2.8	3.4	11.9	25.3	26.5	19.1	4.0		
13,000–13,999:															
Number	1	1	2	10	7	11	21	33	74	72	99	104	3	438	6.0
Percent	0.2	0.2	0.5	2.3	1.6	2.5	4.8	7.5	16.9	16.4	22.6	23.8	0.7		
14,000–14,999:															
Number	2	2	2	2	5	9	7	26	40	124	119	153	13	504	6.9
Percent	0.4	0.4	0.4	0.4	1.0	1.8	1.4	5.1	7.9	24.6	23.6	30.4	2.6		
15,000–15,999:															
Number	1	----	1	3	1	11	7	23	27	50	212	470	12	818	11.1
Percent	0.1	----	0.1	0.4	0.1	1.3	0.9	2.8	3.3	6.1	25.9	57.5	1.5		
16,000–16,999:															
Number	1	----	----	9	6	9	10	15	29	32	144	204	27	486	6.6
Percent	0.2	----	----	1.8	1.2	1.8	2.1	3.1	6.0	6.6	29.6	42.0	5.6		
17,000–17,999:															
Number	1	----	----	1	3	2	7	5	42	99	173	15	18	366	5.0
Percent	0.3	----	----	0.3	0.8	0.5	1.9	1.4	11.5	27.0	47.3	4.1	4.9		
18,000–18,999:															
Number	----	2	2	1	1	12	3	5	14	111	118	156	14	439	6.0
Percent	----	0.5	0.5	0.2	0.2	2.7	0.7	1.1	3.2	25.3	26.9	35.5	3.2		
19,000–19,999:															
Number	----	----	----	1	----	3	1	3	50	47	212	108	43	468	6.4
Percent	----	----	----	0.2	----	0.6	0.2	0.6	10.7	10.1	45.3	23.1	9.2		
20,000 and over:															
Number	----	----	----	2	2	16	1	21	10	112	164	940	75	1,343	18.3
Percent	----	----	----	0.1	0.1	1.2	0.1	1.6	0.8	8.3	12.2	70.0	5.6		
TOTAL:															
Number	152	47	65	106	193	205	214	322	708	1,174	1,657	2,273	233	7,349	100.0
Percent	2.1	0.6	0.9	1.5	2.6	2.8	2.9	4.4	9.6	16.0	22.5	30.9	3.2		

Table 13.—Table for estimating the distribution of 3-axle, tractor-semitrailer combinations (2–S1) grouped by recorded empty weight by groups of probable registered gross vehicle weights

Recorded empty weight of tractor (pounds)	Registered gross combination weight (pounds)															Total number	Percent of total
	Under 18,000	18,000–19,999	20,000–21,999	22,000–23,999	24,000–25,999	26,000–27,999	28,000 29,999	30,000–31,999	32,000–35,999	36,000–39,999	40,000–44,999	45,000–49,999	50,000–54,000	55,000–59,999	60,000–64,999	65,000 and over		
Under 5,000:																		
Number	------	30	5	6	18	12	10	17	20	36	60	3	1	------	1	------	219	3.5
Percent	------	13.7	2.3	2.7	8.2	5.5	4.6	7.8	9.1	16.4	27.4	1.3	0.5	------	0.5	------		
5,000–5,999:																		
Number	------	25	15	39	48	16	33	41	96	69	104	13	11	1	2	------	513	8.2
Percent	------	4.9	2.9	7.6	9.4	3.1	6.4	8.0	18.7	13.5	20.3	2.5	2.1	0.2	0.4	------		
6,000–6,999:																		
Number	------	24	14	14	80	34	55	83	361	229	314	25	17	------	------	------	1,250	19.9
Percent	------	1.9	1.1	1.1	6.4	2.7	4.4	6.7	28.9	18.3	25.1	2.0	1.4	------	------	------		
7,000–7,999:																		
Number	------	17	28	19	37	21	31	151	329	310	404	27	31	5	2	------	1,412	22.4
Percent	------	1.2	2.0	1.3	2.6	1.5	2.2	10.7	23.3	22.0	28.6	1.9	2.2	0.4	0.1	------		
8,000–8,999:																		
Number	2	8	9	9	40	29	36	54	107	183	572	77	68	10	6	1	1,211	19.2
Percent	0.2	0.7	0.7	0.7	3.3	2.4	3.0	4.5	8.9	15.1	47.2	6.4	5.6	0.8	0.5	(1)		
9,000–9,999:																		
Number	------	2	6		11	10	11	31	77	92	437	88	134	19	15	------	937	14.9
Percent	------	0.2	0.6	0.4	1.2	1.1	1.2	3.3	8.2	9.8	46.7	9.4	14.3	2.0	1.6	------		
10,000–10,999:																		
Number	------	------	------	1	4	3	6	8	31	40	173	41	66	14	35	2	424	6.7
Percent	------	------	------	0.2	0.9	0.7	1.4	1.9	7.3	9.4	40.8	9.7	15.6	3.3	8.3	0.5		
11,000–11,999:																		
Number	------	------	------	1	3	2	4	8	19	22	108	13	40	11	24	2	257	4.1
Percent	------	------	------	0.3	1.1	0.8	1.6	3.1	7.4	8.6	42.0	5.1	15.6	4.3	9.3	0.8		
12,000 and over:																		
Number	------	------	------	------	------	------	1	1	------	6	16	14	8	6	19	1	72	1.1
Percent	------	------	------	------	------	------	1.4	1.4	------	8.3	22.2	19.5	11.1	8.3	26.4	1.4		
TOTAL:																		
Number	2	106	77	93	241	127	187	394	1,040	987	2,188	301	376	66	104	6	6,295	100.0
Percent	(1)	1.6	1.2	1.5	3.8	2.0	3.0	6.3	16.5	15.7	34.8	4.8	6.0	1.0	1.7	0.1		

[1] Less than 0.1 percent.

Table 14.—Table for estimating the distribution of 4-axle, tractor-semitrailer combinations (2–S2) grouped by recorded empty weight by groups of probable registered gross vehicle weights

Recorded empty weight of tractor (pounds)	Registered gross combination weight (pounds)														Total number	Percent of total
	Under 24,000	24,000–25,999	26,000–27,999	28,000–29,999	30,000–31,999	32,000–35,999	36,000–39,999	40,000–44,999	45,000–49,999	50,000–54,999	55,000–59,999	60,000–64,999	65,000–69,999	70,000 and over		
Under 5,000:																
Number	2	1	------	------	------	------	------	3	3	2	------	1	------	1	13	0.1
Percent	15.4	7.7	------	------	------	------	------	23.1	23.1	15.3	------	7.7	------	7.7		
5,000–5,999:																
Number	3	4	7	2	2	1	1	2	8	17	7	2	------	------	56	0.6
Percent	5.3	7.1	12.5	3.6	3.6	1.8	1.8	3.6	14.2	30.4	12.5	3.6	------	------		
6,000–6,999:																
Number	8	17	3	2	7	7	23	32	76	64	60	10	------	------	309	3.5
Percent	2.6	5.5	1.0	0.6	2.3	2.3	7.4	10.4	24.6	20.7	19.4	3.2	------	------		
7,000–7,999:																
Number	1	3	2	3	10	8	15	43	35	127	91	38	------	1	377	4.3
Percent	0.3	0.8	0.5	0.8	2.7	2.1	3.9	11.4	9.3	33.7	24.1	10.1	------	0.3		
8,000–8,999:																
Number	3	1	4	1	1	9	22	61	58	291	269	117	1	1	839	9.5
Percent	0.4	0.1	0.5	0.1	0.1	1.1	2.6	7.3	6.9	34.7	32.1	13.9	0.1	0.1		
9,000–9,999:																
Number	4	3	2	1	11	14	30	60	87	573	784	361	13	2	1,945	22.0
Percent	0.2	0.2	0.1	0.1	0.5	0.7	1.5	3.0	4.5	29.5	40.3	18.6	0.7	0.1		
10,000–10,999:																
Number	5	2	1	2	3	6	5	34	47	439	1,124	515	66	5	2,254	25.4
Percent	0.2	0.1	(1)	0.1	0.1	0.3	0.2	1.5	2.1	19.5	49.9	22.9	2.9	0.2		
11,000–11,999:																
Number	------	------	1	------	4	2	3	19	10	197	652	274	77	4	1,243	14.0
Percent	------	------	0.1	------	0.3	0.2	0.2	1.5	0.8	15.9	52.5	22.0	6.2	0.3		
12,000–12,999:																
Number	3	1	2	------	------	------	1	12	19	92	759	260	45	10	1,204	13.6
Percent	0.3	0.1	0.2	------	------	------	0.1	1.0	1.6	7.6	63.0	21.6	3.7	0.8		
13,000–13,999:																
Number	------	3	------	------	------	------	1	13	10	20	231	91	49	3	421	4.8
Percent	------	0.7	------	------	------	------	0.2	3.1	2.4	4.8	54.9	21.6	11.6	0.7		
14,000 and over:																
Number	------	------	------	------	------	------	------	1	8	21	84	68	10	7	199	2.2
Percent	------	------	------	------	------	------	------	0.5	4.0	10.6	42.2	34.2	5.0	3.5		
TOTAL:																
Number	29	35	22	11	38	47	101	280	361	1,843	4,061	1,737	261	34	8,860	100.0
Percent	0.3	0.4	0.2	0.1	0.4	0.5	1.1	3.2	4.1	20.8	45.9	19.6	3.0	0.4		

[1] Less than 0.1 percent.

Table 15.—Table for estimating the distribution of 5-axle, tractor-semitrailer combinations (3–S2) grouped by recorded empty weights, by groups of probable registered gross vehicle weights

Recorded empty weight of tractor (pounds)	Registered gross combination weight (pounds)							Total number	Percent of total
	Under 50,000	50,000–54,999	55,000–59,999	60,000–64,999	65,000–69,999	70,000–74,999	75,000 and over		
Under 12,000:									
Number	136	48	55	197	129	172	5	742	12.7
Percent	18.3	6.5	7.4	26.5	17.4	23.2	0.7		
12,000–12,999:									
Number	27	57	42	215	316	207	8	872	15.0
Percent	3.1	6.6	4.8	24.7	36.2	23.7	0.9		
13,000–13,999:									
Number	12	20	42	164	183	234	5	660	11.3
Percent	1.8	3.0	6.4	24.8	27.7	35.5	0.8		
14,000–14,999:									
Number	11	16	36	199	145	438	13	858	14.7
Percent	1.3	1.9	4.2	23.2	16.9	51.0	1.5		
15,000–15,999:									
Number	2	7	8	167	154	345	45	728	12.5
Percent	0.3	1.0	1.1	22.9	21.1	47.4	6.2		
16,000–16,999:									
Number	2	2	2	93	211	205	34	549	9.4
Percent	0.4	0.4	0.4	16.9	37.3	38.4	6.2		
17,000–17,999:									
Number	1		3	17	37	712	49	819	14.1
Percent	0.1		0.4	2.1	4.5	86.9	6.0		
18,000 and over:									
Number		1	4	18	11	282	257	603	10.3
Percent		0.2	0.7	3.0	6.8	46.7	42.6		
TOTAL:									
Number	191	151	192	1,070	1,216	2,595	416	5,831	100.0
Percent	3.3	2.6	3.3	18.3	20.9	44.5	7.1		

Table 16.—Table for estimating the distribution of 5-axle truck, full-trailer combinations (3–2) grouped by recorded empty weights, by groups of probable registered gross vehicle weights

Recorded empty weight of truck (pounds)	Registered gross combination weight (pounds)						Total number	Percent of total
	Under 60,000	60,000–64,999	65,000–69,999	70,000–74,999	75,000–79,999	80,000 and over		
Under 14,000:								
Number	10	2	3	14	7		36	5.0
Percent	27.8	5.6	8.3	38.9	19.4			
14,000–14,999:								
Number			2	21	7	1	31	4.3
Percent			6.5	67.7	22.6	3.2		
15,000–15,999:								
Number	1	1	5	11	28	1	47	6.6
Percent	2.1	2.1	10.7	23.4	59.6	2.1		
16,000–16,999:								
Number			5	31	57	5	98	13.7
Percent			5.1	31.6	58.2	5.1		
17,000–17,999:								
Number	1		11	52	52	2	118	16.5
Percent	0.8		9.3	44.1	44.1	1.7		
18,000–18,999:								
Number		1	11	87	54	5	158	22.1
Percent		0.6	7.0	55.0	34.2	3.2		
19,000–19,999:								
Number		1	5	75	56	3	140	19.6
Percent		0.7	3.6	53.6	40.0	2.1		
20,000–20,999:								
Number	5			10	40	4	59	8.3
Percent	8.5			16.9	67.8	6.8		
21,000–21,999:								
Number				2	18		20	2.8
Percent				10.0	90.0			
22 000 and over:								
Number				8			8	1.1
Percent				100.0				
TOTAL:								
Number	17	5	42	311	319	21	715	100.0
Percent	2.4	0.7	5.9	43.5	44.6	2.9		

Table 17.—Table for estimating the distribution of 5-axle, tractor-semitrailer full trailer combinations (2-S1-2) grouped by recorded empty weights, by groups of probable registered gross vehicle weights

Recorded empty weight of tractor (pounds)	Registered gross combination weight (pounds)							Total number	Percent of total
	50,000–54,999	55,000–59,999	60,000–64,999	65,000–69,999	70,000–74,999	75,000–79,999	80,000 and over		
Under 10,000:									
Number	1					1		2	3.0
Percent	50.0					50.0			
10,000-10,999:									
Number				1	6	2		9	13.2
Percent				11.1	66.7	22.2			
11,000-11,999:									
Number			1	2	6	4	1	14	20.6
Percent			7.1	14.3	42.9	28.6	7.1		
12,000-12,999:									
Number			1	1	14	9	2	27	39.7
Percent			3.7	3.7	51.9	33.3	7.4		
13,000-13,999:									
Number						7		7	10.3
Percent						100.0			
14,000 and over:									
Number					2	7		9	13.2
Percent					22.2	77.8			
TOTAL:									
Number	1		2	4	28	30	3	68	100.0
Percent	1.5		2.9	5.9	41.2	44.1	4.4		

Table 18.—Empty weight to gross weight ratios of single-unit trucks and tractor-semitrailers, at selected gross vehicle weights

Vehicle type	Ratio of gross vehicle weight to—	
	Empty weight of power unit only	Empty weight of entire vehicle
Single-unit trucks:		
2-axle		
4,000 pounds GVW	1.2	
32,000 pounds GVW	2.7	
3-axle		
22,000 pounds GVW	2.2	
50,000 pounds GVW	2.8	
Vehicle combinations:		
3-axle (2-S1)		
20,000 pounds GVW	3.2	1.3
50,000 pounds GVW	5.5	2.5
4-axle (2-S2)		
30,000 pounds GVW	3.9	1.7
65,000 pounds GVW	5.8	2.8
5-axle (3-S2)		
50,000 pounds GVW	4.0	2.1
75,000 pounds GVW	4.8	2.7

New Publications

HIGHWAY PROGRESS, 1962

Annual Report of the Bureau of Public Roads, Fiscal Year 1962

The Bureau of Public Roads, U.S. Department of Commerce, presents a review of the accomplishments of the Federal-aid highway program and of its many other activities during the fiscal year 1962 in its annual report, *Highway Progress, 1962.*

Included in the 112-page illustrated publication is a descriptive account of the tremendous progress made during fiscal year 1962 on construction of the National System of Interstate and Defense Highways and in improvement of primary highways, secondary roads, and urban arterials under the regular Federal-aid program. Also described is the highway construction work undertaken directly by the Bureau of Public Roads in national forests and parks and on other Federal lands, as well as Public Roads' activities in providing technical assistance to foreign countries to further their program of highway development.

Also reported on at length are the activities and accomplishments of Public Roads in highway planning and design, urban transportation planning, safety, and its extensive and varied research and development program. Included as an appendix in the report are 19 statistical tables covering the progress and activities of the Federal-aid program during the fiscal year 1962.

Highway Progress, 1962, is available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at 35 cents per copy.

Standard Plans for Highway Bridges

The Bureau of Public Roads has recently issued a 4-volume set of *Standard Plans for Highway Bridges (1962)* to replace the 1956 edition of *Standard Plans for Highway Bridge Superstructures.* The new plans are available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., and may be ordered singly for $1.00 each or as a complete set for $4.00: Vol. I—*Concrete Superstructures;* Vol. II—*Structural Steel Superstructures;* Vol. III—*Timber Bridges,* and Vol. IV—*Typical Continuous Bridges.*

The first three volumes are a substantial revision of the 1953 and 1956 editions of this publication and the fourth volume presents additional information. Volumes I and II contain plans for superstructures for simple concrete and steel bridges respectively. Volume III contains plans for substructures and superstructures for timber bridges; and Volume IV contains complete detailed plans for typical 4-span continuous concrete and steel bridges.

These plans will serve as useful guides to State, county, and local highway departments in the development of suitable and economical bridge designs for primary, secondary, and urban highways. The plans provide information sufficiently complete to approach contract drawings as nearly as practicable. For any given bridge location, however, requirements imposed by site conditions will necessitate modification of the plans.

STATUS OF IMPROVEMENT

THE NATIONAL SYSTEM OF INTERSTATE AND DEFENSE HIGHWAYS

SECTIONS OPEN TO TRAFFIC NOVEMBER 1, 1962
ADDITIONAL SECTIONS EXPECTED TO OPEN BY JANUARY 1, 1965
ADDITIONAL SECTIONS EXPECTED UNDER CONSTRUCTION BY JANUARY 1, 1965
TO BE CONSTRUCTED AFTER 1964
U.S. DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS

Estimated Travel by Motor Vehicles in 1961

BY THE CURRENT PLANNING DIVISION
BUREAU OF PUBLIC ROADS

Reported by THEODORE S. DICKERSON
Highway Research Enginee

MOTOR-VEHICLE travel in the United States in 1961 totaled 737.5 billion vehicle-miles, an increase of 2.6 percent over the travel in 1960. The travel data were compiled from information supplied by the State highway departments and toll authorities. Total travel for 1962, based on information for the first 10 months of the year is estimated at 767 billion vehicle-miles, a 4-percent increase over 1961.

The proportions of travel by road system and by vehicle type changed little from 1960 to 1961. Of the 1961 travel, 40 percent was on main rural roads comprising 14 percent of the Nation's total of 3.6 million miles of roads and streets. Another 46 percent of the travel was on urban streets, which comprise only 12 percent of the total mileage. Local rural roads accounted for only 14 percent of the travel but make up 74 percent of the total mileage.

Passenger cars represented 84 percent of the vehicles and accounted for 82 percent of the travel in 1961; trucks and truck combinations accounted for 16 percent of the vehicles and 17 percent of the travel; buses accounted for less than 1 percent of both vehicles and travel.

Average vehicle performance in 1961 differed very little from that reported for 1960. The average motor vehicle traveled 9,648 miles in 1961, almost half of it in cities, and consumed 776 gallons of fuel at a rate of 12.44 miles per gallon. The average passenger car traveled 9,465 miles and consumed 658 gallons of fuel, at a rate of 14.38 miles per gallon. The average commercial bus traveled a little more and the average truck a little less in 1961 than in 1960, but their average rates of fuel consumption did not change appreciably.

The travel and related information for 196 is shown in table 1 by road system and vehicl type. Such data have been reported i PUBLIC ROADS magazine for a number years; the latest, for 1960, appeared in vo 32, No. 1, April 1962, p. 11.

Table 1.—Estimated motor-vehicle travel in the United States and related data for calenda year 1961 [1]

Vehicle type	Motor-vehicle travel					Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural road travel	Local rural road travel	Total rural travel	Urban travel	Total travel			Total	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles/gal.*
Passenger cars [2]	233,011	79,426	312,437	292,120	604,557	63,870	9,465	42,033	658	14.38
Buses:										
Commercial	878	156	1,034	1,812	2,846	75.2	37,846	610	8,112	4.67
School and nonrevenue	627	664	1,291	259	1,550	205.5	7,543	220	1,071	7.05
All buses	1,505	820	2,325	2,071	4,396	280.7	15,661	830	2,957	5.30
All passenger vehicles	234,516	80,246	314,762	294,191	608,953	64,151	9,492	42,863	668	14.21
Trucks and combinations	62,679	20,461	83,140	45,442	128,582	12,291	10,461	16,443	1,338	7.82
All motor vehicles	297,195	100,707	397,902	339,633	737,535	76,442	9,648	59,306	776	12.44

[1] For the 50 States and District of Columbia.
[2] Includes taxicabs; also motorcycles (595,669 registered).

U.S. GOVERNMENT PRINTING OFFICE:1963

PUBLICATIONS of the Bureau of Public Roads

A list of the more important articles in PUBLIC ROADS *and title sheets for volumes 24–31 are available upon request addressed to Bureau of Public Roads, Washington 25, D.C.*

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D.C. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

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REPORTS TO CONGRESS

Factual Discussion of Motortruck Operation, Regulation and Taxation (1951). 30 cents.

Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

Highway Cost Allocation Study:

- First Progress Report, House Document No. 106 (1957). 35 cents.
- Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.
- Final Report, Part VI: Economic and Social Effects of Highway Improvement, House Document No. 72 (1961). 25 cents.

The 1961 Interstate System Cost Estimate, House Document No. 49 (1961). 20 cents.

U.S. HIGHWAY MAP

Map of U.S. showing routes of National System of Interstate and Defense Highways, Federal-aid Primary Highway System, and U.S. Numbered Highway System. Scale 1 inch equals 80 miles. 25 cents.

PUBLICATIONS

Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

America's Lifelines—Federal Aid for Highways (1962). 15 cents.

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Classification of Motor Vehicles, 1956–57 (1960). 75 cents.

Design Charts for Open-Channel Flow (1961). 70 cents.

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1955, $1.00. 1956, $1.00. 1957, $1.25. 1958, $1.00. 1959, $1.00. 1960, $1.25.

Highway Statistics, Summary to 1955. $1.00.

Highway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.

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Hydraulics of Bridge Waterways (1960). 40 cents.

Increasing the Traffic-Carrying Capability of Urban Arterial Streets: The Wisconsin Avenue Study (1962). 40 cents. Appendix, 70 cents.

Landslide Investigations (1961). 30 cents.

Manual for Highway Severance Damage Studies (1961). $1.00.

Manual on Uniform Traffic Control Devices for Streets and Highways (1961). $2.00.

Parking Guide for Cities (1956). Out of print.

Peak Rates of Runoff From Small Watersheds (1961). 30 cents.

Road-User and Property Taxes on Selected Motor Vehicles, 1960. 30 cents.

Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1958: a reference guide outline. 75 cents.

Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP–61 (1961). $2.25.

Standard Plans for Highway Bridges (1962):

- Vol. I— Concrete Superstructures. $1.00.
- Vol II Structural Steel Superstructures. $1.00.
- Vol. III —Timber Bridges. $1.00.
- Vol. IV—Typical Continuous Bridges. $1.00.

The Identification of Rock Types (revised edition, 1960). 20 cents.

The Role of Aerial Surveys in Highway Engineering (1960). 40 cents.

Transition Curves for Highways (1940). $1.75.

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JUNE 1963

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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Woodrow Wilson Memorial Bridge over the Potomac River connects Virginia and Maryland. An important link in the Capital Beltway, this bridge provides access to the District of Columbia from the south.

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Vol. 32, No. 8 *June 196*

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Use of funds for printing this publication has been approved by t Director of the Bureau of the Budget, March 6, 1961.

Contents of this publication may be re-printed. Mention of source is requested.

Comparison of Two Methods for Preloading Electronic Scales

By RUSSELL E. PUCKETT, Assistant Professor of Electrical Engineering, and JAMES E. GOVER, Research Assistant in Instrumentation, University of Kentucky [1]

To meet the need for collecting and analyzing large volumes of data requisite to highway planning, highway engineers are using new tools such as electronic weighing systems and other electronic instrumentation systems. This article reports on another phase of a research project on electronic weighing systems. The purpose of such a system is to weigh vehicles in the traffic stream without stopping them or interfering with their travel. In doing this the electronic device must be able to detect and record an electric analog of the load applied to the system, without including variable factors such as the vibration and oscillation of the weighing platform. To achieve stability of the platform, different methods of preloading have been devised. Although these methods of preloading the platform achieve the desired stability, they also reduce the overall sensitivity of the measuring system to imposed loads. This article presents an analysis of two preloading methods and shows their relative effectiveness in maintaining system sensitivity while achieving mechanical stability of the weighing platform.

Introduction

ELECTRONIC scales are a relatively new addition to the types of tools used by the ighway engineer. Many agencies concerned ith planning and maintenance of adequate ystems of highways have established and valuated different methods for obtaining load ata. The electronic scale has attracted conderable attention because electronic measurng techniques permit the solution of many roblems related to weighing moving vehicles. Many systems have been developed in which lectronic techniques are used to measure and ecord both static and dynamic loads. Some dvantages of the electronic systems, as pposed to mechanical scales, are the sensiivity that permits detection of extremely mall loads and observation of high-speed henomena, which occur too fast for visual bservation, and the small physical spaces equired for electronic load detectors (*1*).[2]

Instrumentation of electronic load-measurng systems serves two purposes; (1) detection f an electric analog of the load being measured nd (2) rapid acquisition of a tremendous olume of data and its reduction to summary orm for use in quick analysis. Some of the roblems inherent in producing an accurate nalog of a load being measured with an lectronic instrumentation system are disussed in this article. Specifically, data are resented relative to the system being devel-ped and tested by the University of Kentucky n a research project that is sponsored jointly y the U.S. Bureau of Public Roads and the Kentucky Department of Highways. Purose of this research project is to develop a lependable system for weighing vehicles, articularly trucks, as they roll along the ighway (*2*).

Conclusions

From an analysis of the two methods of preloading the platform of an electronic weighing device with coil springs and with steel rods, it has been concluded that preloading with coil springs affords better stability and has little effect on the sensitivity of the measuring system.

To achieve maximum benefit from an electronic weighing system, the largest possible output must be obtained. The output analog of the applied load always will be relatively small and anything that reduces or tends to reduce it should be avoided or made as ineffective as possible (*3*).

This research demonstrated that heavy coil springs, which may be stretched a considerable distance in relation to motion of the weighing platform, afforded stability of the platform while maintaining the overall sensitivity of the measuring system. When steel rods were used for preloading, however, the sensitivity of the system was reduced to an extent that negated the effectiveness of the rods in stabilizing the platform. Therefore preloading a dynamic platform with coil springs is more advantageous than using steel rods for this purpose.

Load Measuring

To record dynamic load data, some means must be provided for detecting the load. A transducer capable of accepting the load data as a mechanical force and converting it to an electric analog may be used in an electronic weighing system. The input to an electronic scale is a physical force proportional to the applied load. The output of the transducer should be an electric analog of the load. Many types of load detectors have been used but the strain gage load cell has been employed in the research reported here. A complete description of the strain gage load cell, the principle of its operation, and the recording instrumentation are detailed in references *2* and *4*. The basic construction of a typical load cell is shown in figure 1.

Commercial designs have been developed for electronic weighing systems that use a platform supported at its four corners by load cells; figure 2 shows a typical installation. The platform is set level with the road surface to measure the axle loads of trucks as they roll over it. The output of the load cells is proportional to the weight applied to the platform by the truck wheels (*4*). Many problems have been encountered when this type of system has been used to measure the axle load of trucks in motion. The principal problem has been related to leveling the platform on its four supports so as to prevent its tipping and thus causing unbalanced loads on the four load cells. Some degree of success has been obtained in overcoming this problem (*4*). By preloading the platform with tension turnbuckles and steel rods, it has been leveled and the tendency for it to oscillate upon application of load has been greatly reduced.

To eliminate horizontal movement of the weighing platform without reducing the sen-

---

[1] These authors are members of a research team at the University of Kentucky conducting an investigation of problems related to weighing a moving vehicle. The research project is sponsored by the U.S. Bureau of Public Roads and the Kentucky Department of Highways. David K. Blythe, Head of the Civil Engineering Department is project director and John A. Dearinger, Assistant Professor of Civil Engineering, is assistant project director.

[2] References indicated by italic numbers in parentheses are listed on page 185.

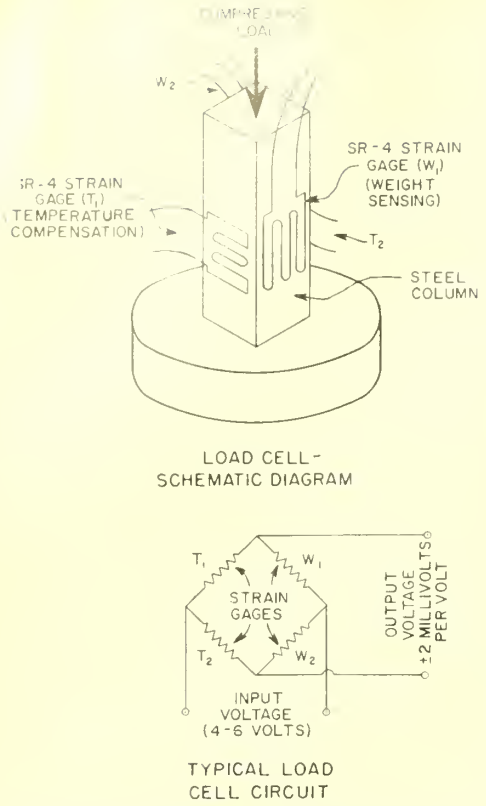

Figure 1.—Construction of typical load cell.

sitivity of the system to vertical loads, the Research Organization for Roadbuilding in West Germany has developed and built a broken-bridge design that is less subject to vibration than the platform supported on four corners. The broken-bridge design is shown in figure 3; this system has two, narrow steel boxes that rest on the foundation of the platform's supporting structure. Load cells that convert the load into an electric analog are located beneath the center joint connecting the halves of the bridge (*5*, *6*).

Platform Used at University of Kentucky

A broken-bridge platform, based on the German design, was developed for the research project at the University of Kentucky and installed on the University's farm for use in different tests of the electronic weighing system. An exterior view of the platform in place at the test site is shown in figure 4. Two types of preloading devices steel rods and turnbuckles or heavy coil springs—were attached to the platform to stabilize the system to the applications of dynamic loads. Tests were made to determine the performance of the system with each of the preloads. An interior view of the scale pit, including the installation of steel rods and turnbuckles between the platform and the bottom of the scale pit, is illustrated in figure 5. The us of heavy coil springs to preload the weighin device is shown in figure 6.

Both preloading methods reduced platfor oscillation under loading. However, the over all sensitivity of the recording instrumentatio was reduced more when the steel rods wer used for preloading than when the coil spring were used. The output of the load cell supporting the platform was greatly reduce when preloading was accomplished by th steel rods but the output remained practicall at its "no-preload" value for any load whe the springs were used for preloading the sys tem. The two preloading methods and a analysis of the differences in sensitivity of th system are discussed in detail in the followin paragraphs.

Preload Effect on System Sensitivity

The force diagram of figure 7 represents th broken-bridge weighing platform. P_o repre sents the preload applied to the platform b each of the two preloading members, one a each end of the platform. For this analysis it was assumed that no bending of the platfor occurs between the load cells and the point of application of the preload. It was als assumed that a linear relationship of the pre loading members exists between their elonga tion and the force applied. Based on thes assumptions, the preload was represented b the expression:

$$P_o = ky_o \qquad (1$$

Where,

P_o = preload
y_o = elongation of preloading members
k = constant of proportionality.

Figure 8 is a force diagram of the platfor as it appears when an axle load of $2W$ wa applied symmetrically to the platform. Suc an application causes compression of the loa cells to some distance Δy. Application of th axle load to the platform reduced the tensil force in each of the preloading members b $k\Delta y$. Summation of the vertical forces on th platform showed that the load carried by eac load cell may be expressed as:

$$P_1 = P_o - k\Delta y + W \qquad (2$$

Where,

P_1 = load on load cell
P_o = preload
k = constant of proportionality
Δy = distance load cells compressed
W = applied weight (actual load).

When the preload was first applied, th instrumentation was adjusted to its zer position. When an axle load was applied t the platform, the instrumentation indicate an analog of the difference between the initia preload value and the load carried by the loa cell. This difference is the value of $P_1 - P$ which may be expressed in terms of th distance of compression of the load cel equation (2), as:

$$P_1 - P_o = W - k\Delta y \qquad (3$$

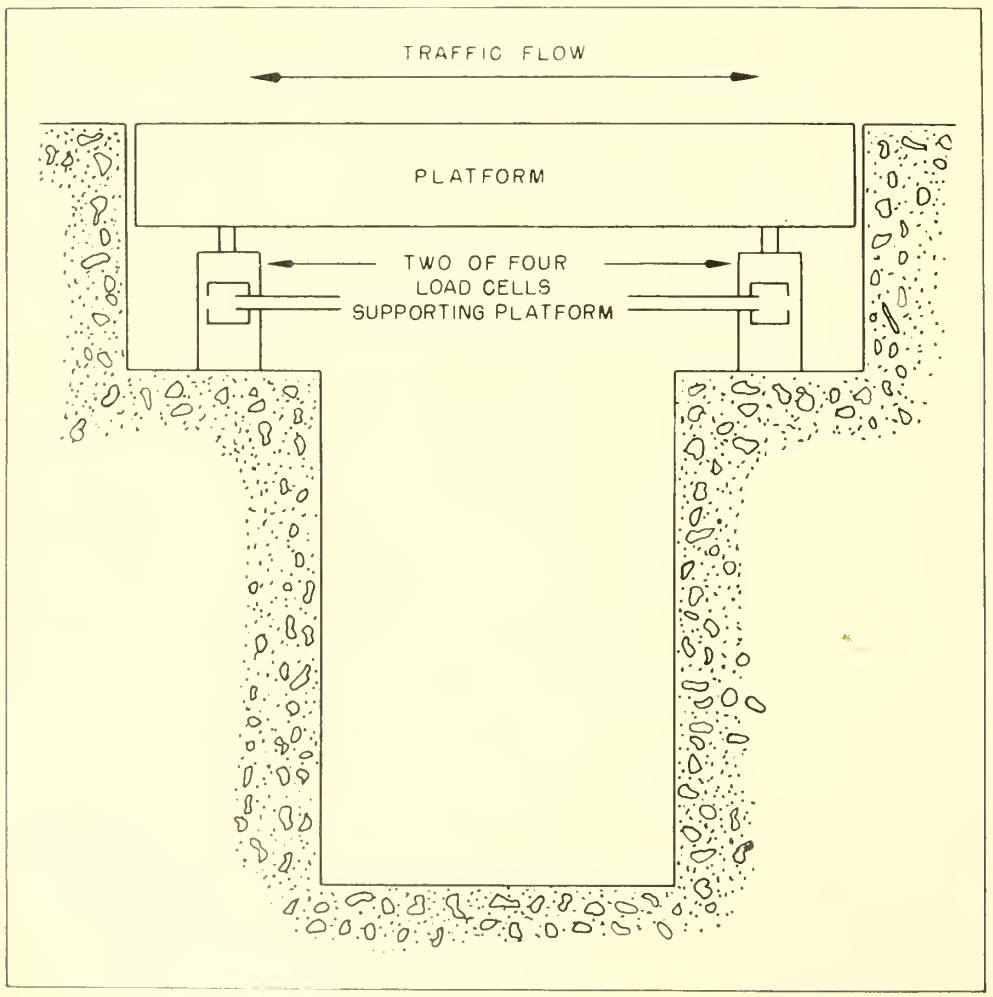

Figure 2.—Installation of platform supported at four points.

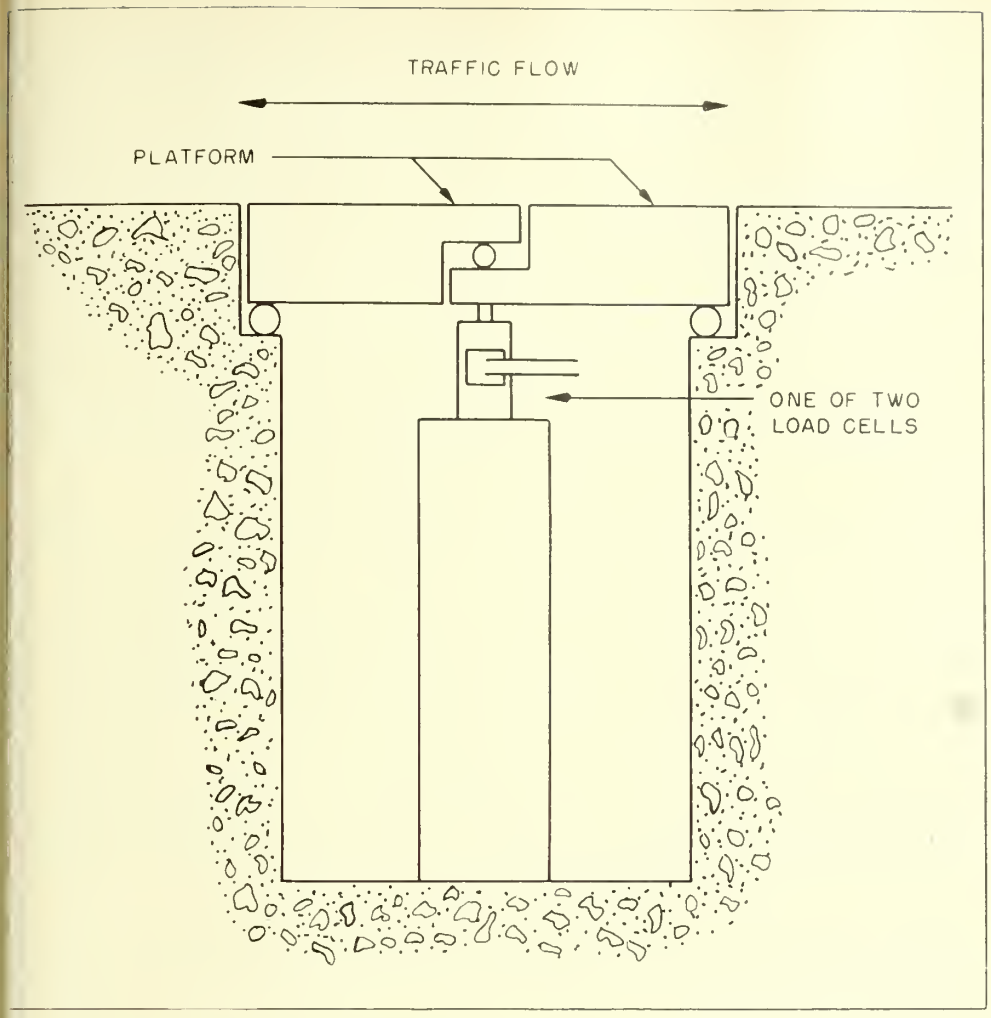

Figure 3.—Installation of broken-bridge platform.

Equation (3) shows that the analog of the eight indicated on the instrumentation will e in error by $k\Delta y$. Therefore, this factor ould be kept as small as possible so that the alog of the applied load will be more nearly presentative of the actual load, W. The agnitude of Δy is predetermined by the size the load applied to the platform and the sic sensitivity of the load cell. For the pe of load cell used on this project, Δy was proximately 0.010 inch for a 50,000-pound ad. This shows that changes in the value k are required to increase the overall nsitivity of the system.

Because the initial preload equals ky_o, the alue of k must not be made so small that ie product ky_o is too small to permit com- etion of the original purpose of preloading ie platform—stabilization of the platform id reduction of vertical oscillation. This quirement suggests use of a device that will low a large value for y_o and a small value r k, thereby keeping the product $k\Delta y$ small.

Comparison of Methods

Preloading during these tests was achieved two ways: (1) heavy springs were mounted etween the platform and the bottom of the scale pit, and (2) steel rods were tied to the platform and anchored to the bottom of the scale pit. Both devices were adjustable to permit changing the value of preload. Because of the physical construction of the pit and the platform, both the steel rods and the coil springs were limited in length.

An extension of equation (2) shows the effect of the factor $k\Delta y$ in the two methods of preload. Using equation (1), the effect of the factor may be written as:

$$P_1 = ky_o - k\Delta y + W \tag{4}$$

or as:

$$P_1 = k(y_o - \Delta y) + W \tag{4}$$

When the coil springs were used for preloading, the initial elongation of the preload member could be made large in comparison with any compression distance during the application of a load, and can be expressed as:

$$y_o >> y \tag{5}$$

A "worst case" check may be made for equation (5). Assume that the springs are preloaded at an elongation of 6 inches and that a 25,000-pound axle load is applied to the platform. If the load were placed symmetrically on a platform supported by two load cells, each capable of being compressed 0.010 inch at 50,000-pound load, the compression distance of a cell would be:

$$\Delta y = \frac{12{,}500}{50{,}000} \times 0.010 \text{ inch} = 0.0025 \text{ inch}.$$

Because y_o equals 6 inches, equation (5) is valid.

When preloading is accomplished by using coil springs, equation (5) may be approximated as:

$$y_o - \Delta y \approx y_o \tag{6}$$

Based on this approximation, equation (4) may be stated as one or the other of the three following expressions.

$$P_1 = ky_o + W,$$
$$P_1 = P_o + W,$$
$$P_1 - P_o = W.$$

Thus, when coil springs are used for preloading, the applied load can be recorded as an electronic analog that has no serious error caused by the preload. However, when steel rods are used for preloading the platform, the approximation developed in equation (6) is not valid because the characteristics of steel rods prevent their stretching any significant distance. Consideration of equation (4) shows that when steel rods are used for preloading significant error will be reflected in the analog for any load applied to the platform.

Estimating Reduction of Sensitivity

Further manipulation of equation (3) simplifies the comparison of the two methods of preload tested. The load carried by the load cell may be written as:

$$P_1 - P_o = C\Delta y \tag{7}$$

Where,

C = basic sensitivity of the load cell.

Substitution of equation (7) in equation (3) yields, either

$$P_1 - P_o = W - k\,\frac{P_1 + P_o}{C}$$

or

$$(P_1 - P_o) = \frac{W}{1 + \frac{k}{C}} \tag{8}$$

Equation (8) is illustrated in figure 9. The slope of the line defining actual sensitivity is

$$\frac{1}{1 + \frac{k}{C}}$$

compared with the maximum possible slope of unity when no preload is being used or when $k = 0$.

Equation (8) indicates that the overall sensitivity of the measuring system is reduced by any preload, and the sensitivity depends only upon the method of application of the preload; that is, the value of k. The difference between the load applied and its analog, when no preload was used, is given by the separation

Figure 4.—Broken-bridge scale installed at the test site.

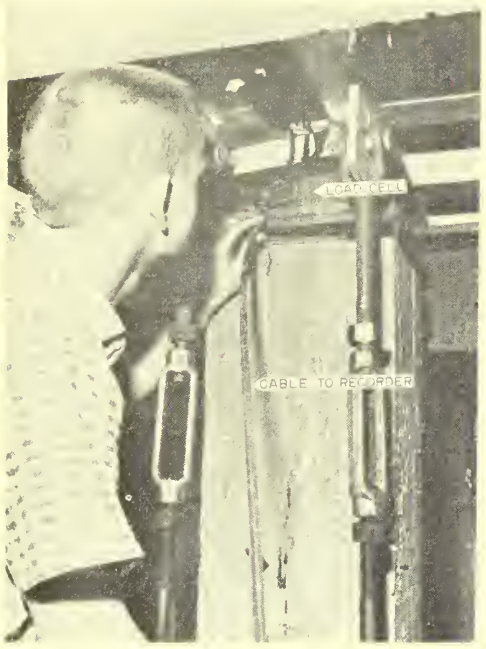

Figure 5.—Steel rods and turnbuckles used for preloading the platform.

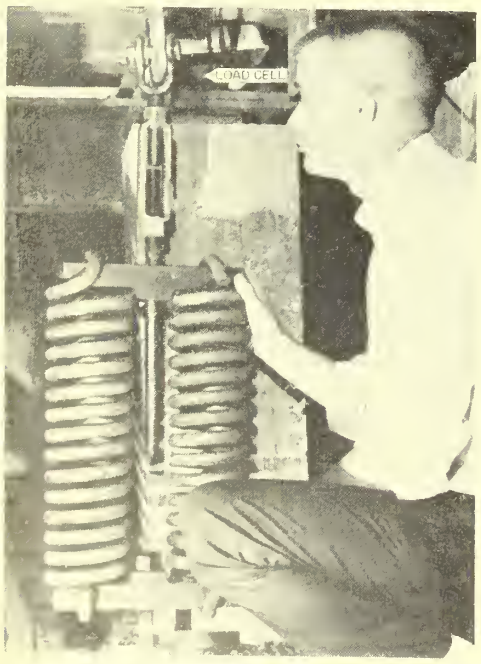

Figure 6.—Preloading accomplished with springs.

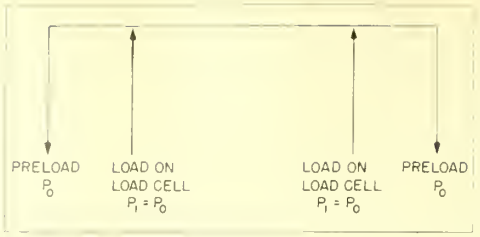

Figure 7.—Force diagram of platform with initial preload—no load applied.

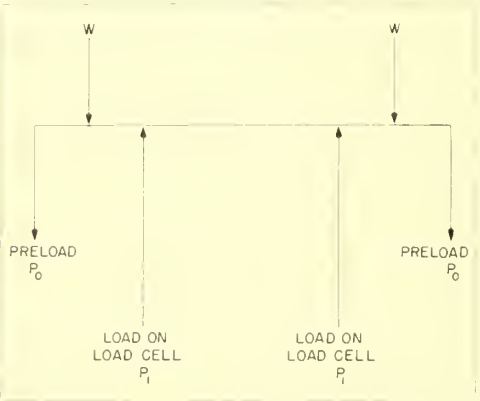

Figure 8.—Force diagram of platform with preload—load applied.

of the straight line in figure 9. Because the lines are straight, any error of an analog of load will be a fixed percentage of the load it represents, regardless of the magnitude of the load. Also, the error will be dependent upon the magnitude of the preload being used. This reduction of the system's sensitivity will be a constant, provided the value of k remains constant.

The graph of figure 9 has been plotted for only one value each for k and C. Other values would, of course, yield different curves. The separation of the two lines would increase for larger values of k. This again emphasizes the necessity for keeping the value of k as small as possible; the separation represents the reduction of the overall sensitivity of the instrumentation system.

Experimental Comparison of Preload Methods

In order to check the validity of the foregoing analysis of preload methods, a 3-axle truck was used for controlled tests. The amount of preload was based on the static weight of the front axle of this truck. The preload—either coil springs or steel rods—was applied to the platform in increments of 50 percent of the static weight. Weights were recorded at different speeds of the truck and for various amounts of preload that ranged up

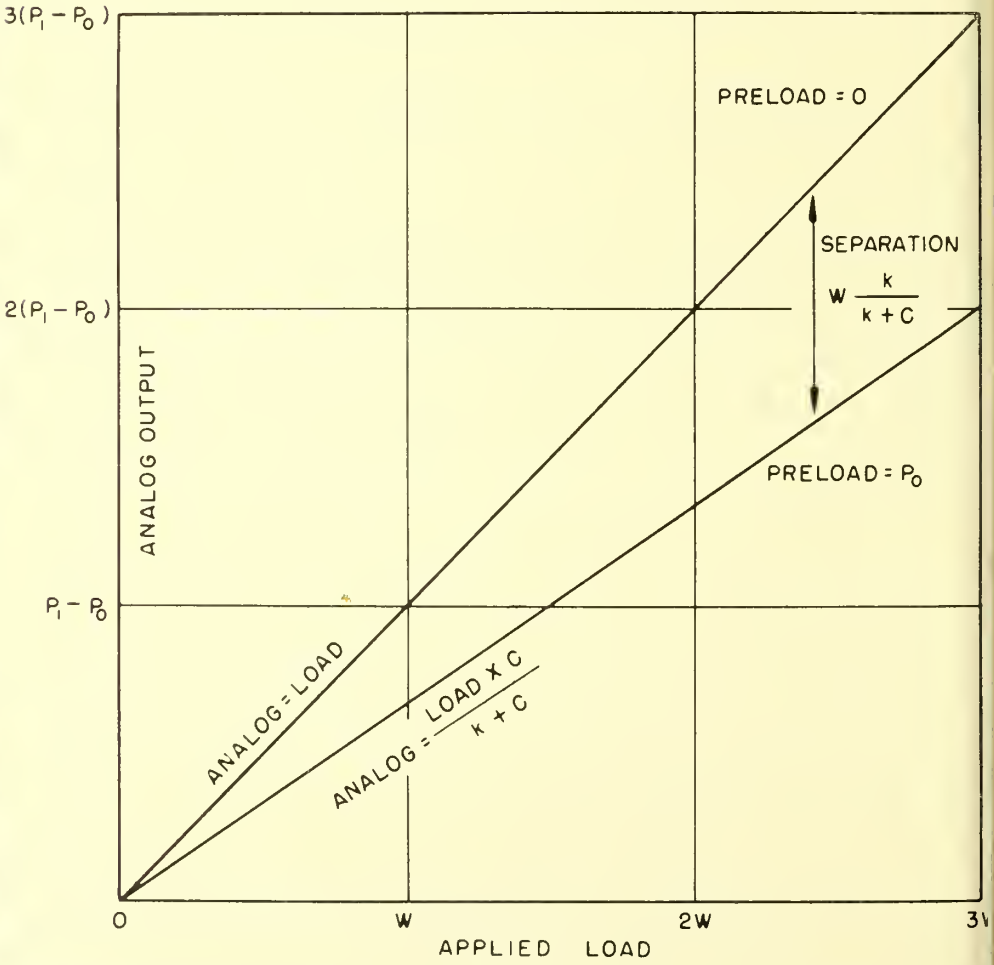

Figure 9.—Theoretical analog output as a function of applied load—preload constant.

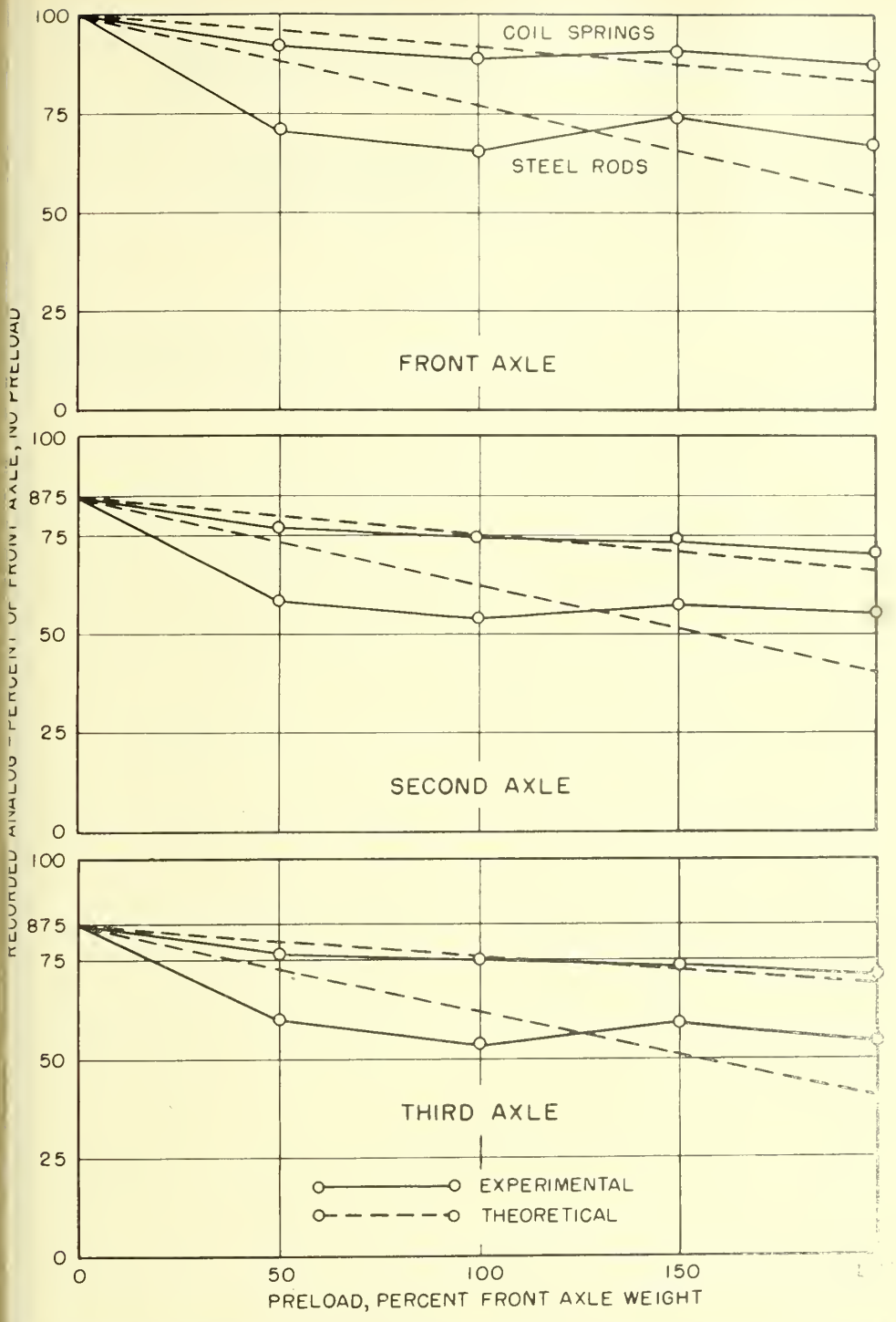

Figure 10.—Experimental results that show reduction of sensitivity caused by preload.

to 200 percent of the front-axle weight. Typical results for each of the 3 axles obtained from these tests are shown by figure 10.

Lines representing the theoretical sensitivity curves of the system have been included in figure 10 for comparison with the results of the experimental tests. Although the experimental curves do not coincide with the theoretical predictions, they have the same general trend in slope. Part of the difference in the curves has been attributed to some bending of the platform between the load-cell supports and the point at which the preload was applied to the platform. Other factors, such as changes in the value of k of the preload devices and inaccurate measurement of the preload being used, also may have accounted for some of the difference between the curves. The curves for preloading with springs and those for preloading with steel rods have different slopes; this difference shows the effect of the different values of k.

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Potential Applications of Spectroscopy in the Highway Testing Laboratory

BY THE
MATERIALS RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

By [1] **WOODROW J. HALSTEAD, Supervisory Chemis**
and BERNARD CHAIKEN, Chemi

Highway testing laboratories are confronted with an increasing volume and variety of construction materials to be tested for compliance with chemical or mineralogical requirements. In addition, many new and complex materials used in construction, such as plastics, synthetic polymers, surface active agents, coatings, etc. are too complex to be effectively analyzed by ordinary chemical means. The potential advantages of using spectroscopy methods to provide more rapid and accurate analyses of materials than is otherwise possible by standard chemical procedures are discussed in this article. The suitability of spectroscopic techniques to determine the nature of complex materials, which cannot be practicably analyzed by chemical methods, is also discussed. The article includes general estimates of cost and time factors involved for several of the more useful techniques.

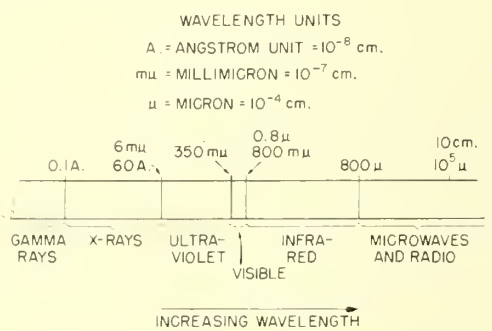

Figure 1.—Electromagnetic spectrum.

Introduction

THE EVER-INCREASING numbers and types of materials submitted to the highway laboratory for tests have created serious problems for the testing engineer and not the least of these is the need for rapid methods of chemical analysis. Everyone is desperately longing for the ultimate gadget that will permit them to insert a sample in one end, push a button, wait 30 seconds, and have a complete report come out the other end.

This ultimate is, of course, a pipe-dream that most likely will never be completely realized. However, in some areas of analyses, this dream is closer to reality than might be suspected. The rapid development of arc spectroscopy in the last 10 to 15 years has provided the means of determining, within minutes, the absence or presence of as many as 70 elements in a metal. The alkali content of cements and other materials can be determined in minutes by flame spectroscopy as opposed to the 3 or 4 days required for an analysis by older classical methods. Not quite ready, but definitely on the horizon within the next few years, are X-ray fluorescence techniques that will permit a complete chemical analysis of cement and similar materials in about 30 minutes, sometimes less. A general evaluation of new spectroscopic tools to determine if and where they can be put to work in the highway testing laboratory is presented in this article.

Summary

It is difficult to make a definite statement as to what type of instruments would be economical for all highway testing laboratories. The type and number of samples to be tested, whether special investigations are to be conducted as well as control tests; the time factors involved, all enter into the decision as to whether spectroscopic instruments should be purchased. In most laboratories, a flame photometer and a manually operated ultraviolet-visible light absorption spectrophotometer, at a total cost of about $3,000, ea

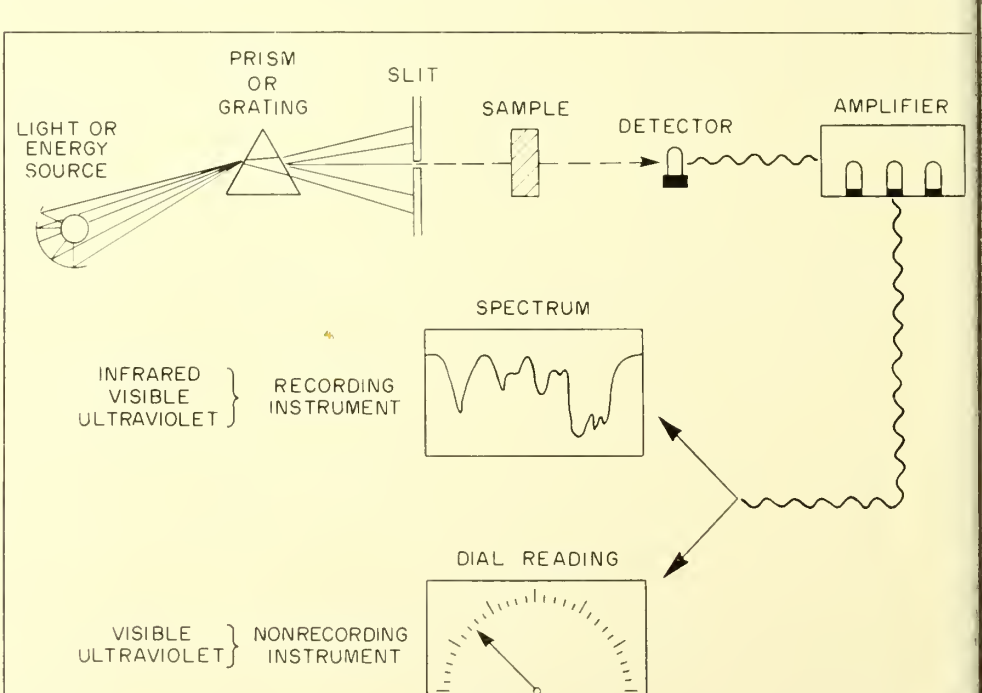

Figure 2.—Schematic diagram of an absorption spectrophotometer.

[1] This article is based on a talk presented by Mr. Halstead at the 37th annual open meeting of the New Jersey, New York, and New England States Testing Engineers Association, Boston, Mass., Nov. 1961.

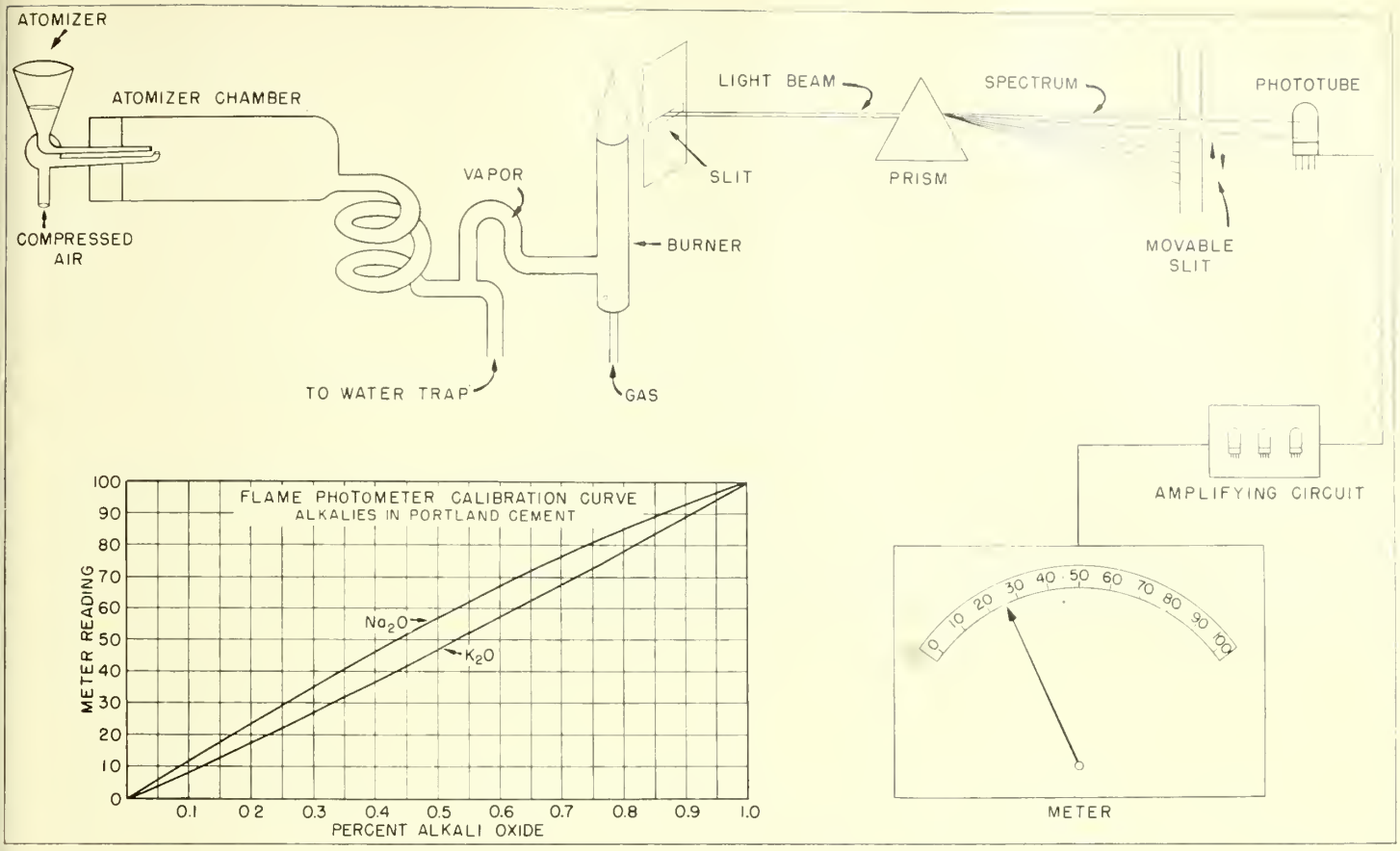

Figure 3.—Schematic diagram of a flame photometer.

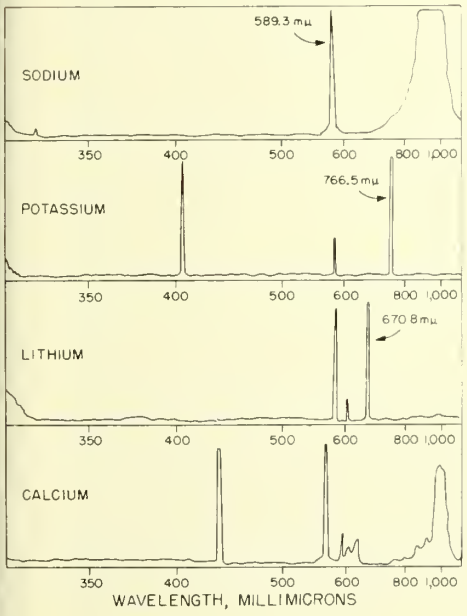

Figure 4.—Flame spectra of elements.

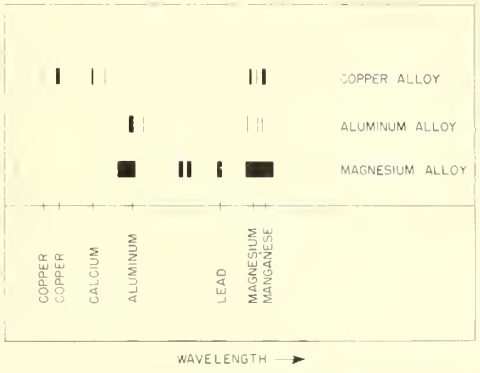

Figure 5.—Type of spectra obtained with an arc emission spectograph.

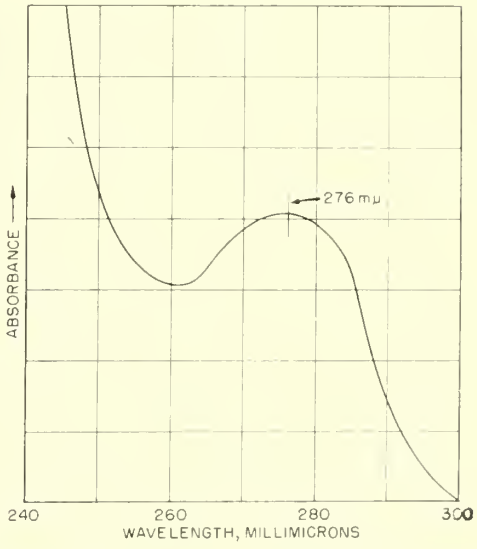

Figure 6.—Ultraviolet absorption curve of phthalic acid. (Prepared from Bureau of Standards Sample 84e).

asily be justified for: (1) routine testing of cements, rocksalt, and other materials for lkalies, (2) for work on concrete admixtures, 3) for identification of phosphorus in fer-ilizer, titantium in paint pigments, and lloying constituents in metals for which pectrophotometer methods are available.

An infrared spectrophotometer can be justified if the laboratory is regularly concerned with controlling the uniformity of proprietory concrete admixtures, rubber or synthetic water-stops, traffic paint vehicles, epoxy resins, silicones, herbicides, and similar complex organic materials. An instrument costing about $5,000 is suitable for accomplishing these purposes.

Arc and spark spectrographs can be justified only if very large volumes of metals must be analyzed. Considerable specialized accessory equipment is needed for this type of installation. The laboratory must be air conditioned, the humidity must be controlled, and a photographic dark room and metal working tools, such as lathes, belt polishers, etc., must be provided. A minimum cost for adequate facilities would probably be about $40,000.

X-ray diffraction is very useful for identifying soils and solving problems related to the base or foundation of the highway, but it has limited application to specification control work. Nuclear magnetic resonance, at pres-

Figure 7.—Relation between concentration and ultraviolet absorbance of a lignosulfonate retarder.

ent, is essentially a research tool and its potential applications have not been considered in this discussion.

For the future, perhaps in 5 or 10 years, X-ray fluorescence may be developed to provide rapid and accurate analyses of major as well as minor constituents of many materials such as cements, alloys, and steels. At the present time, however, X-ray fluorescence also should be considered a research tool. The present cost of this equipment is from $25,000 to $40,000.

Spectroscopy is a rapidly developing science and entirely new techniques are constantly being developed. For example, a relatively new instrument called "atomic absorption spectroscopy" has recently been introduced and it may make flame photometers obsolete. However, its potentialities for highway materials have not yet been evaluated.

Table 1 provides a guide to those interested in the present applicability of spectroscopic tools to the control of standard highway materials. The table shows only those instruments that have immediate applicability to the materials listed. The data presented should be considered as approximations only, and types of materials or analyses given are not necessarily complete.

Principles of Spectroscopy

The principle upon which all spectroscopic methods are based is the utilization and measurement of radiant energy in the electromagnetic spectrum. Figure 1 is a schematic diagram of this spectrum. All of the types of radiation illustrated—gamma, X-rays, ultraviolet, and visible light; infrared energy; and radio waves—are qualitatively identical in that they are light energy waves moving in accordance with the same basic law, which is,

Wavelength × Frequency = Speed of Light.

This means that frequency varies inversely with wavelength; that is, the shorter wavelengths of energy, such as X-rays and gamma rays, have greater frequencies. Frequency is also equated to photon energy: the greater the frequency, the greater the photon energy. Electromagnetic energy is utilized in two ways for practical analytical purposes: (1) it is absorbed by the test sample, hence the term *absorption* spectroscopy, and (2) it is emitted by the test sample, hence the term *emission* spectroscopy.

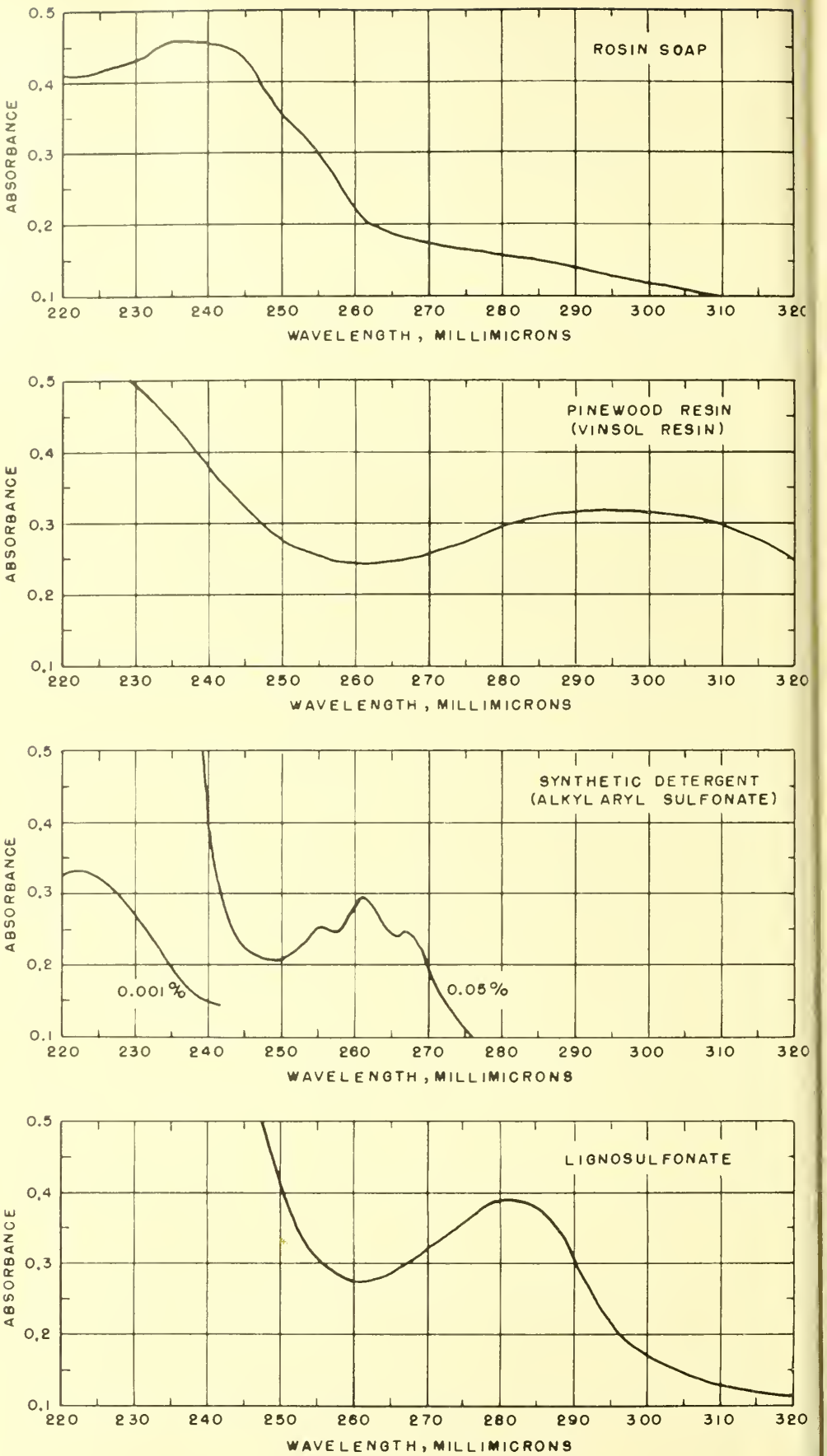

Figure 8.—Ultraviolet spectra of concrete admixtures.

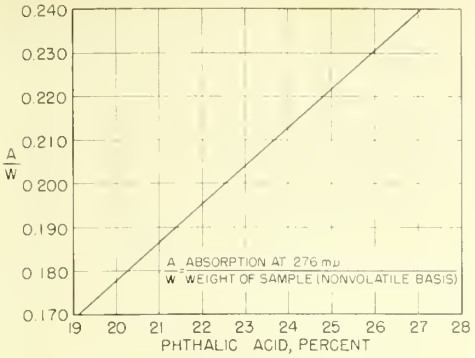

Figure 9.—Absorbance vs. concentration of phthalic acid solutions.

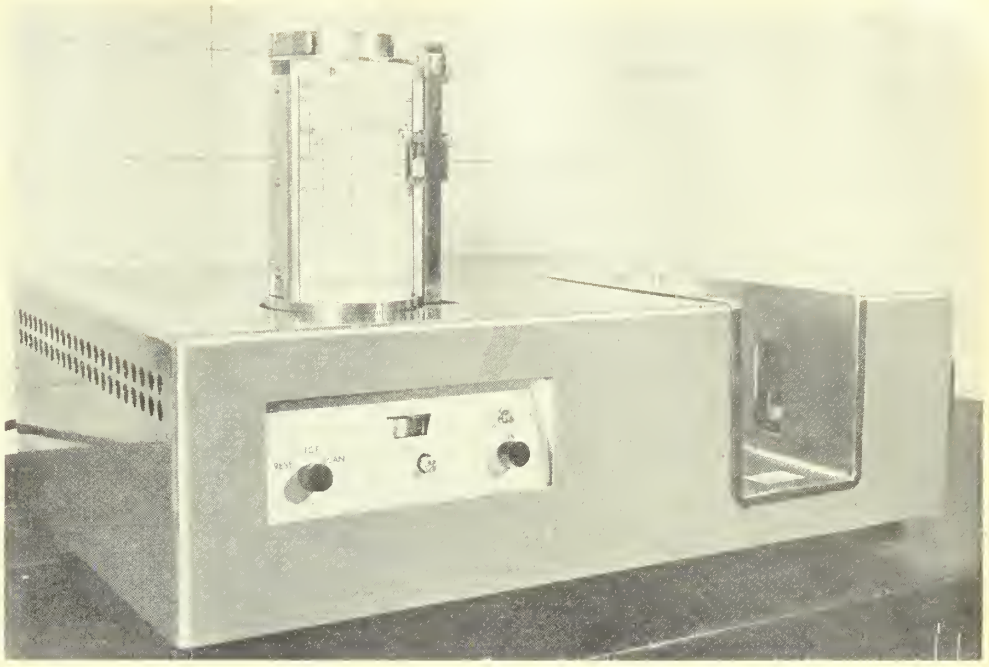

Figure 10.—An infrared spectrophotometer.

Absorption spectroscopy

In absorption spectroscopy, visible light or some other form of radiant energy is allowed to pass through a sample, and the amount of light or energy absorbed at different wavelengths is recorded. A schematic diagram of the basic principles involved is shown in figure 2. As illustrated, the beam of light from the source is dispersed or spread out into its different wavelengths by either a prism or grating. Such radiation, either before or after dispersion, is permitted to pass through a sample and the amounts of energy absorbed by the sample at the different wavelengths are measured. The total energy absorbed for all wavelengths is recorded as a continuous spectrum, but that absorbed at an individual wavelength can be indicated as a meter reading, also shown in figure 2.

Instruments such as ultraviolet, visible light, and infrared spectrophotometers belong to the general class used for absorption spectroscopy. In a less rigorous sense, nuclear magnetic resonance, X-ray diffraction, and X-ray fluorescence equipment are also absorption spectrometers.

Emission spectroscopy

In emission spectroscopy, analysis of the material is based on the fact that many chemical elements, when sufficiently heated in a flame or electric arc, emit radiation or give off light having specific characteristics. When the emitted radiation from a particular sample is resolved or spread out by the dispersing medium of the instrument—either a prism or grating—an array of sharp, distinct, and separate lines is obtained. The positions of the different lines—their wavelengths—are related to the kinds of atoms present, and the intensities of the lines are proportional to their concentration. The flame photometer, typical of instruments used in the field of emission spectroscopy, is shown schematically in figure 3. The following list summarizes the more important techniques used for absorption and emission spectroscopy.

Absorption spectroscopy.—X-ray diffraction, X-ray fluorescence, ultraviolet, visible light, infrared, and nuclear magnetic resonance.

Emission spectroscopy.—Flame, and arc and spark.

Some of these techniques have little or no potential use in a highway testing laboratory but others are already being used, and still other techniques have considerable potential applications if the instrumentation can be further perfected. The general principles of several types of instruments that appear to have significant potential application to the analysis of highway materials are discussed in detail in the following portion of this article.

Flame Photometry

The flame photometer is perhaps the most familiar spectroscopic instrument to the highway engineer. It has been employed with excellent success during the past 15 years for the determination of alkalies, particularly sodium and potassium oxides, in cements and other substances. The principle involved in flame photometry requires that an element to be determined must be suitably excited by a flame so that it will emit characteristic radiations. The sample is dissolved in a suitable solvent if it is not already liquid, it is then atomized into a suitable gas flame for excitation, and a measurement is made of the resultant intensity of emitted radiation. The types of spectra that can be obtained with a special recording flame spectrophotometer are illustrated in figure 4. As shown, the response for specific elements is usually very sharp. However, commercial instruments are generally designed for quantitative work by meter readings, thus a wavelength is selected for each element at which the response for the desired element is strongest or at which interference from other substances would be at a minimum. The final result is a meter reading indicative of the intensity of emitted radiation, and it therefore is a measure of the concentration of the element sought.

Applications and instrumentation

Flame photometry is applicable only to inorganic constituents, either in solid or liquid samples. It is particularly applicable to the alkali metals—sodium, lithium, and potassium. Each of these elements emits characteristic radiation at a relatively low temperature such as that produced by an ordinary air-propane flame. For sufficient excitation of other elements, higher flame temperatures are required. In addition to being useful for alkali determinations in cement, flame photometers can be used to determine easily the amount of sodium in rocksalt ice-removal agents, and the potash content in fertilizers used for roadside seeding. The instrument has been used in exploratory work in connection with stripping tests of asphaltic materials; lithium salt was utilized as a tracer to quantitatively determine the degree of stripping.

(Continued on page 192)

Motor Vehicle Size and Weight Limits

A comparison of State legal limits of motor vehicle sizes and weights with standards recommended by the American Association of State Highway Officials is given in the table on pages 190–191. The statutory limits reported in this tabulation, prepared by the Bureau of Public Roads as of December 31, 1962, have been reviewed for accuracy by the appropriate State officials.

Statutory limits are shown for width, height, and length of vehicles; number of towed units; maximum axle loads for single and tandem axles; and maximum gross weights for single-unit truck, truck-tractor semitrailer combinations, and other combinations.

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STATE LEGAL MAXIMUM LIMITS OF MOTOR VEHI

Prepared by the E

Line	State	Width inches[1]	Height ft.-in	Length-feet[1]				Numbered of towed units[3]			Axle load-pounds				
				Single unit		Truck tractor semi-trailer	Other combination	Semi-trailer	Full trailer	Semi-trailer and full trailer	Single		Tandem		Type o
				Truck	Bus						Statutory limit	Including statutory enforcement tolerance	Statutory limit	Including statutory enforcement tolerance	
1	Alabama	96	13-6	35	40	50	NP	1	NP	NP	18,000	19,800	36,000	39,600	Table
2	Alaska	96	12-6	35	[19]40	60	60	1	1	2	18,000		32,000		Table-tire
3	Arizona	96	13-6	40	40	65	65	1	1	2	18,000		32,000		Table
4	Arkansas	96	13-6	35	40	50	50	1	1	NP	18,000	[7]18,500	32,000	32,500	Spec. mo
5	California	96	13-6	35	[9]35	60	65	NR	NR	NR	18,000		32,000		Table
6	Colorado	[11]96	[12]13-6	35	40	60	[10]60	1	2	2	18,000		36,000		Formula-
7	Connecticut	102	12-6	50	50	50	NP	1	NP	NP	22,400	22,848	36,000	36,720	Spec. lim.
8	Delaware	96	13-6	40	42	55	60	1	1	NP	20,000		36,000		Table-sp
9	District of Columbia	96	12-6	40	40	50	50	1	1	NP	[46]22,000		38,000		Table
10	Florida	96	13-6	[14]35	40	55	55	1	1	NP	20,000	22,000	40,000	44,000	Table
11	Georgia	96	13-6	[15]39	[15]45	50	50	1	1	NP	18,000	20,340	36,000	40,680	Spec. mo
12	Hawaii	108	13-0	40	40	55	65	1	1	2	24,000		32,000		Formula[1]
13	Idaho	[11]96	14-0	35	[19]40	60	65	1	1	2	[20]18,000		[20]32,000		Table[20]
14	Illinois	96	13-6	42	42	[21]55	60	1	1	2	[21]18,000		32,000		Spec. lim.
15	Indiana	[11]96	13-6	36	40	50	[25]50	1	1	2	[23]18,000	[23]19,000	[23]32,000	[23]33,000	Spec. lim.
16	Iowa	96	13-6	35	[19]40	50	50	1	[24]1	3	18,000	18,540	32,000	32,960	Table
17	Kansas	96	13-6	35	[19]40	50	50	1	1	NP	18,000		32,000		Table
18	Kentucky	96	[12]13-6	[26]35	[26]35	[27]50	[27]50	1	NP	NP	18,000	[28]18,900	32,000	[28]33,600	Spec. lim.
19	Louisiana	96	13-6	35	[19]40	55	60	1	1	NP	18,000		32,000		Axle lim.-
20	Maine	96	[30]12-6	55	55	55	55	1	1	NP	[30]22,000		[30]32,000		Table-tire
21	Maryland	[18]96	[6]12-6	55	55	55	[40]55	NR	NR	NR	22,400		[31]40,000		Formula
22	Massachusetts	96	NR	35	[19]40	50	NP	1	NP	NP	22,400		36,000		Table-spe
23	Michigan	96	13-6	35	40	55	55	1	1	2	[33]18,000		[34]32,000		Axle lim.
24	Minnesota	96	13-6	40	40	50	50	1	1	NP	18,000		32,000		Table
25	Mississippi	96	13-6	35	40	55	55	1	1	NP	18,000		28,650	[35]32,000	Table-tire
26	Missouri	96	12-6	35	40	50	50	1	1	2	18,000		32,000		Table
27	Montana	[11]96	13-6	35	40	60	60	1	1	[37]2	18,000		32,000		Table
28	Nebraska	96	13-6	40	40	60	60	1	1	[c]2	18,000	18,900	32,000	33,600	Table
29	Nevada	96	NR	NR	NR	NR	NR	NR	NR	NR	18,000	18,900	32,000	33,600	Table
30	New Hampshire	96	13-6	35	[39]40	50	50	NR	NR	NR	22,400		36,000		Tables-sp
31	New Jersey	[41]96	[42]13-6	35	[39]35	50	[40]50	1	1	NP	22,400	23,520	32,000	33,600	Spec. lim
32	New Mexico	[41]96	13-6	40	40	65	65	1	1	2	21,600		34,320		Table
33	New York	96	[6]13-0	35	[43]35	50	50	1	1	NP	22,400		36,000		Formula
34	North Carolina	96	[6]12-6	35	[19]40	[43]50	[43]55	1	1	NP	18,000	19,000	36,000	38,000	Spec. lim
35	North Dakota	[44]96	[44]13-6	[14]35	[19]40	60	60	1	1	2	18,000		32,000		Formula
36	Ohio	96	13-6	35	[19]40	50	60	1	NR	NR	19,000		31,500		Formula
37	Oklahoma	96	13-6	35	45	[45]60	[45]60	1	1	NP	18,000		32,000		Table
38	Oregon	96	[12]13-6	35	[35]40	[29,35]55	[35]65	1	1	[35]2	[47]18,000		[47]32,000		Table[48]
39	Pennsylvania	96	[6]12-6	25	40	[46]50	[40]50	1	1	NP	22,400	23,072	36,000	37,080	Spec. lim
40	Puerto Rico	96	12-6	35	40	50	50	1	1	NP	N5		N5		Spec. lim
41	Rhode Island	102	12-6	40	40	50	50	1	1	NP	22,400		N5		Spec. lim
42	South Carolina	96	13-6	[14]35	[19]40	55	[59]60	1	1	NP	20,000		32,000		Table
43	South Dakota	96	13-6	35	40	60	60	1	1	2	18,000		32,000		Table
44	Tennessee	96	[6]12-6	35	40	50	50	1	[54]1	NP	18,000		32,000		Table
45	Texas	96	13-6	35	40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Table
46	Utah	96	14-0	45	45	60	60	NR	NR	NR	18,000		[64]33,000		Table[65]
47	Vermont	96	12-6	50	50	50	50	1	1	NP	NS		N5		Spec. lim
48	Virginia	96	13-6	35	40	50	50	1	1	NP	18,000		[57]32,000		Table
49	Washington	96	13-6	35	[19]40	60	[58]65	1	1	[58]2	18,000		32,000		Table
50	West Virginia	96	[6]12-6	35	[19]40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Table
51	Wisconsin	96	13-6	35	40	50	[67]50	1	1	NP	18,000	[60]19,500	30,400	32,000	Table[61]
52	Wyoming	96	13-6	40	40	65	65	1	1	2	18,000		32,000	[62]36,000	Table
	AASHO Policy	96	12-6	35	[19]40	50	60	1	1	NP	18,000		32,000		Table
Number of States	Higher	3	45	18	33	26	9	5	6	24	31		30		Formula
	Some	49	7	34	14	26	13	47	42	28	21		21		Table
	Lower	0	0	0	5	0	30	0	4	0	0		1		Specifie

NP—Not permitted. NR—Not restricted. NS—Not specified.

[1] Various exceptions for farm and construction equipment; public utility vehicles; house trailers; urban, suburban, and school buses; haulage of agricultural and forest products; at wheels of vehicles for safety accessories, on designated highways, and as administratively authorized.

[2] Various exceptions for utility vehicles and loads, house trailers and mobile homes.

[3] When not specified, limited to number possible in practical combinations within permitted length limits; various exceptions for farm tractors, mobile homes, etc.

[4] Legally specified or established by administrative regulation.

[5] Computed under the following conditions to permit comparison on a uniform basis between States with different types of regulation:

A. Front axle load of 8,000 pounds.

B. Maximum practical wheelbase within applicable length limits:

(1) Minimum front overhang of 3 feet.

(2) In the case of a 4-axle truck-tractor semitrailer, rear overhang computed as necessary to distribute the maximum possible uniform load on the maximum permitted length of semitrailer to the single drive-axle of the tractor and to the tandem axles of the semitrailer, within the permitted load limits of each.

(3) In the case of a combination having 5 or more axles, minimum possible combined front and rear overhang assumed to be 5 feet, with maximum practical load on maximum permitted length of semitrailer, subject to control of loading on axle groups and on total wheelbase as applicable.

C. Including statutory enforcement tolerances as applicable.

[6] Auto transports 13 feet 6 inches; Maryland also allows 13 feet 6 inches for vehicles loaded with hay or straw, or carrying flat glass.

[7] Does not apply to combinations of adjacent load-carrying single axles.

[8] 56,000 pounds on load-carrying axles, exclusive of steering-axle load.

[9] On specific routes in urban or suburban service under special permit from P.U.C. 40 feet, also 3-axle buses with turning radius less than 45 feet without restriction.

[10] Except 3-unit combinations may use up to 65 ft. combinations on certain highways designated by the Department of Highways.

[11] Buses 102 inches on highways of surfaced width of least 20 feet or otherwise as administratively authorized.

[12] On class AA, or designated highways, 12 ft. 6 in. on other highways; log and lumber trucks limited to 12 ft. 6 in. on all highways in Oregon.

[14] Legal limit 60,000 pounds, axle spacing 27 feet or more.

[15] Three-axle vehicles 40 feet.

[15] Truck 39.55 feet; bus 45.20 feet.

[16] 63,280 pounds maximum, except on roads un

[17] 700 (L + 40) when L is 18' or less; 800 (L + tures with span of 20' or over.

[18] Vehicles loaded with tobacco hogsheads—10

[19] Less than three axles 35 feet.

[20] Special limits for vehicles hauling timber or products including livestock; single axle 18,900 pou axles permitted 66,000 pounds maximum at 21-foot o at 43-foot axle spacing.

[21] On designated highways; 16,000 pounds on

[22] Without tandem axles 45,000 pounds.

[23] On designated highways; single axle 22,400 all excesses of weight under one or more limitations front or steering axle.

[24] Towing agent must be registered for total gr registered as "Farm trailer."

[25] 60 ft. in special cases: Illinois, auto transp tractor semitrailers on designated major routes.

[26] On designated highways; trucks 26.5 feet on

[27] Class AA highways; 45 feet on other highwa

[28] Class AA highways only.

[29] Maximum gross weight on Class A highways

[30] Including load 14 feet; various exceptions fo

[31] Tandem axles spaced less than 48 inches ap

[32] Subject to axle and tabular limits.

[33] Single axle spaced less than 9 feet from nea

[34] On designated highways only and limited to

[35] On designated highways only.

[36] Administrative regulation—32,000 pounds all and 5 is 28 ft. or more.

[37] Semitrailer and semitrailer converted to full

[38] Dual-drive axles; otherwise 40,000 pounds.

[39] Or as prescribed by P.U.C.

[40] Exception for poles, pillings, structural unit

ND WEIGHTS COMPARED WITH AASHO STANDARDS

ds, December 31, 1962.

imit		Specified maximum gross weight—pounds[1]						Practical maximum gross weight—pounds[1]						Line
Applicable to:		Truck		Truck-tractor semitrailer			Other combination	Truck		Truck-tractor semitrailer			Other combination	
y up l es	Total wheel base only	2-axle	3-axle	3-axle	4-axle	5-axle		2-axle	3-axle	3-axle	4-axle	5-axle		
	X							27,800	47,600	47,600	60,010	64,650	NP	1
er 18'	Over 18'	36,000	50,000	50,000	72,000	76,800	76,800	26,000	40,000	44,000	58,000	72,000	76,800	2
er 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,800	3
								26,500	40,500	45,000	59,000	65,000	65,000	4
er 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,000	5
	X	30,000	46,000					26,000	44,000	44,000	62,000	76,000	76,000	6
		32,000	50,000	50,000	60,000	60,000	NP	30,848	44,720	51,000	61,200	61,200	NP	7
X		30,000	46,000	48,000				28,000	44,000	48,000	64,000	73,280	73,280	8
X					70,000	70,000	70,000	30,000	46,000	52,000	[54]68,000	[54]70,000	[54]70,000	9
	X							30,000	52,000	52,000	65,200	73,095	73,095	10
							63,280	28,340	48,680	48,680	63,280	63,280	63,280	11
X								32,000	38,800	56,000	64,000	72,000	80,000	12
X								26,000	40,000	44,000	58,000	73,280	76,800	13
		36,000	[22]41,000	45,000	59,000	72,000	72,000	26,000	40,000	44,000	58,000	72,000	72,000	14
							72,000	27,000	41,000	45,000	59,000	[23]73,000	[23]73,000	15
X								26,540	40,960	45,080	59,500	73,280	73,280	16
X								26,000	40,000	44,000	55,470	73,280	73,280	17
		27,000	42,000	42,000	59,640	73,280	NP	27,000	42,000	42,000	59,640	73,280	73,280	18
								26,000	40,000	44,000	58,000	72,000	76,000	19
X		32,000	[30]51,800	51,800	60,050	70,550	70,550	30,000	40,000	51,800	62,040	70,550	70,550	20
	X			65,000	65,000	65,000	65,000	30,400	48,000	52,800	65,000	65,000	65,000	21
		[32]46,000	[32]60,000	[32]60,000	[32]73,000	[32]73,000	NP	30,400	44,000	52,800	66,400	73,000	NP	22
								26,000	[35]40,000	44,000	[35]58,000	[35]66,000	[35]102,000	23
X						[36]73,280		26,000	40,000	44,000	58,000	[36]72,000	72,500	24
X								26,000	[35]40,000	44,000	59,000	[35]64,650	[35]64,650	25
X								26,000	40,000	44,000	55,470	64,650	64,650	26
der 18'	Under 18'							26,000	40,000	44,000	58,000	72,000	76,000	27
X		36,000	54,000	54,000	71,146	71,146	71,146	26,780	41,200	45,320	59,740	73,280	73,280	28
der 18'	Under 18'							26,900	41,600	45,800	60,500	75,200	76,800	29
	X	33,400	[38]47,500	52,800	66,400			30,400	44,000	52,800	66,400	66,400	66,400	30
		30,000	40,000	60,000	60,000	60,000	60,000	31,500	41,600	55,040	63,000	63,000	63,000	31
der 18'	Over 18'							29,600	42,320	51,200	63,920	76,640	86,400	32
	X					65,000	65,000	30,400	44,000	52,800	65,000	65,000	65,000	33
		31,500	46,200	46,200	65,100	65,100	65,100	27,000	46,000	46,000	65,100	65,100	65,100	34
der 18'	Over 18'							26,000	40,000	44,000	60,000	[44]64,000	[44]64,000	35
	X							27,000	39,500	46,000	58,500	71,000	78,000	36
X								26,000	40,000	44,000	58,000	72,000	73,280	37
der 18'	Over 18'					[48]76,000	[48]76,000	26,000	40,000	44,000	58,000	72,000	[48]76,000	38
		33,000	47,000	50,000	60,000	60,000	62,000	31,072	45,080	51,500	61,800	61,800	63,860	39
														40
		[50]36,000	[51]44,000	[52]50,000	[53]60,000	60,000	88,000	30,400	44,000	50,000	60,000	60,000	88,000	41
X								28,000	40,000	48,000	60,000	66,839	71,115	42
X								26,000	40,000	44,000	58,000	72,000	73,280	43
	X							26,000	40,000	44,000	58,000	61,580	43,500	44
X								26,900	41,600	45,800	60,500	75,200	75,600	45
X								36,000	51,000	54,000	69,000	79,900	79,900	46
		[55a]32,000	[55b]55,000	[55c]52,800	[55d]66,400	[55d]66,400	[55d]66,400	[55a]32,000	[55b]55,000	[55c]52,800	[55d]66,400	[55d]66,400	[55d]66,400	47
X						70,000	70,000	26,000	40,000	44,000	60,000	70,000	70,000	48
der 18'	Over 18'	28,000	36,000	46,000	60,000	68,000	72,000	26,000	36,000	44,000	60,000	68,000	72,000	49
X					[66]70,000	[66]70,000	[66]70,000	26,900	41,600	45,800	57,844	63,840	63,840	50
X								27,500	40,000	47,000	59,500	73,000	73,000	51
X								26,000	44,000	44,000	62,000	73,950	73,950	52
X								26,000	40,000	44,000	55,470	61,490	71,900	
20	18							30	27	29	49	49	32	
								21	21	21	2	0	0	
								0	3	1	0	2	19	

hority 56,000 pounds maximum.
r than 18'; 900 (L + 40) on highways having no struc-

res, concentrates, oggregates, and agricultural
00 pounds, gross weight table: vehicle with 3 or 4
with 5 or more axles permitted 79,000 pounds maximum

e 36,000 pounds; tolerance of 1,000 pounds on total of
ss weight; depending upon the placing of 9000# on the '

combination of vehicles except former having trailer

-trucks pulling house trailers only; Oregon, truck

ther highways.

Class B highways 30,000 pounds.
orest products and construction materials.
eight limitation of 36,000 pounds.

13,000 pounds.
combination; otherwise 26,000 pounds.

tandum axles provided the distance between axles 2

o dolly.

., permitted 70 feet.

[41] On designated highways 102 inches. Body restricted to 96", additional 6" for tires only.
[42] Trackless trolleys and buses 7 passengers or more, P.S.C. certificate 40 feet.
[43] Including front and rear bumpers.
[44] Vehicles in excess may be operated under special permit obtained in advance from the Department of Motor Vehicles; in North Dakota, from State Highway Truck Regulatory Department.
[45] Auto transports only, by special permit only, otherwise 50 feet.
[46] Any single axle exceeding 18,000 lbs. shall be equipped with 4 properly inflated tires.
[47] Logging vehicles permitted 7-foot wheelbase tolerance, 19,000-single axle, 34,000-pound tandem axle.
[48] Governs gross weight permitted on highways designated by resolution of State highway commission.
[49] Single unit truck with 4 axles permitted 60,000 pounds.
[50] Axles spaced less than 6 feet 32,000 pounds; less than 12 feet 36,000 pounds; 12 feet or more gross weight governed by axle limit.
[51] Single vehicle with 3 or more axles spaced less than 16 feet 40,000 pounds; less than 20 feet 44,000 pounds; 20 feet or more governed by axle limit.
[52] Tractor semitrailer with 3 or more axles spaced less than 22 feet 46,000 pounds; not less than 27 feet 50,000 pounds.
[53] Limited to 3,500 pounds.
[54] Several bridges posted at lower limits.
[55] On Interstate Routes: a. 30,000 lbs.; b. 40,000 lbs.; c. 50,000 lbs.; d. 60,000 lbs.
[56] Where truck-tractor was properly registered in Pennsylvania as of December 31, 1961, 55 feet.
[57] Vehicles registered before July 1, 1956, permitted limits in effect January 1, 1956, for life of vehicle.
[58] Three-unit combinations and full truck and full trailer combinations on designated highways.
[59] House trailers only, otherwise 55 feet.
[60] Axle load 21,000 pounds on 2-axle trucks hauling peeled or unpeeled forest products cut crosswise or transporting milk from farm to market but not over Interstate System.
[61] On Class A highways. All axles of a vehicle or combination—73,000 pounds maximum. Wheel, axle, axle group and gross vehicle weights on Class B highways are 60% of weights including tolerance authorized for Class A highways.
[62] Based on ruling of Attorney General.
[63] Weight limits to be established by administrative regulations.
[64] For axle spacing under 8 feet.
[65] Weights are established on axle spacing of the extreme of any group.
[66] Only on certain highways, or portions thereof, designated by State Roads Commissioner, and consistent with Congressional action.
[67] Mobile house and towing vehicle—50 feet except Noon to Midnight Sundays and other designated holidays.

Potential Applications of Spectroscopy in the Highway Field

(Continued from p. 189)

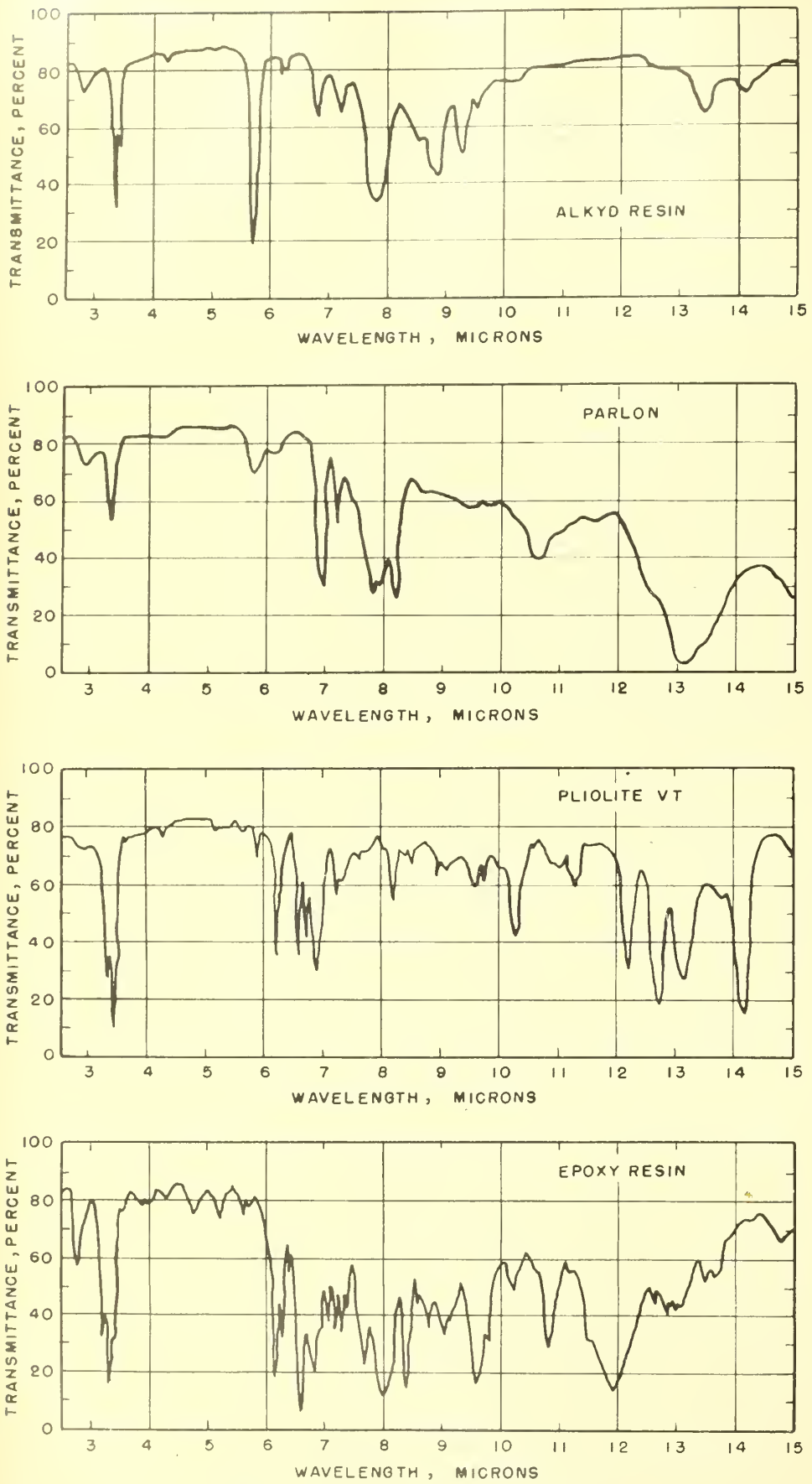

Figure 11.—Infrared spectra of paint resins.

In recent reports, flame photometric methods have been developed for the individual determinations of manganese, magnesia, and the alkalies in a single solution of cement (*1*, *2*, *3*).[2] However, these procedures have not been developed to an acceptable level for routine application.

Much testing time can be saved by use of the flame photometer. For example, the classical gravimetric method for determining the alkalies in portland cements requires 3 to 4 days. With flame photometry, 10 to 15 samples can easily be run in less than a day. This great saving in time for such analyses, coupled with the relatively moderate cost of the instrument, makes the purchase of such equipment worthwhile for analyzing even a relatively small number of samples, 20 to 30 a year.

Flame photometers are available either as a separate tool or as an attachment to manually operated ultraviolet-visible light absorption spectrophotometers. The cost of flame photometers ranges from $500 to $2,500 depending on the refinements of the circuit. The presently available filter-type instruments, which are generally lower priced, are not sufficiently accurate for the determination of the alkali content of cements. The present ASTM and AASHO methods for cement alkalies require a light-dispersing prism or grating and have been written around a specific instrument. However, revisions in these methods are now being made so that any commercial type of instrument may be used provided it produces results within a prescribed degree of accuracy for tests on standard cements of certified alkali content.

Arc and Spark Spectroscopy

The classic example of emission spectroscopy is the arc or spark spectrograph. In principle arc and spark spectroscopy is similar to flame photometry, except that the sample is vaporized and excited by an electric arc or spark rather than by a flame. Here again, only the inorganic elements present can be determined and identified. The arc or spark spectograph is one of the most highly sensitive tools available for analytical work. In its operation, a small sample is burned between electrodes in an electric arc or spark and the sample's spectrum is recorded on a photographic plate or other sensing device. An inspection of the resultant pattern of spectral lines serves to confirm the presence or absence of about 70 of the chemical elements. The positions of the lines are used to qualitatively identify the elements that are present, and the intensity of each line is used as a quantitative measure of the amount of each element present. A typical spectrum of the type obtained on a photographic plate is shown in figure 5.

[2] References indicated by italic numbers in parentheses are listed on page 198.

Quantitative results from arc and spark spectrographs are obtained by comparing the intensities of the lines of the test sample with the intensities of lines from standard samples of known composition. In order to obtain suitable accuracy it is necessary to make a careful standardization of the photographic emulsion, match the unknown with a standard of similar composition, and obtain accurate measurements of the intensities. Under these conditions, accuracy is in the range of 5 to 15 percent of the amount of the constituent present, provided the amount of the constituent does not exceed 10 percent of the sample. Accurate quantitative determinations cannot be made for major constituents that make up more than 10 percent of the sample.

Newer types of spectrographic instruments, which have a photoelectric system for quantitative read out, have much greater accuracy and precision for quantitative work (about percent of the element present). However, these instruments are very expensive and are limited in scope when used for qualitative applications. Such instruments are most suitable for a large volume of repetitive determinations of the same elements in the same types of materials.

Application and instrumentation

Many materials of interest to the highway engineer can be analyzed by an arc and spark spectrograph. These include minor constituents in aluminum and other metallic alloys, minor constituents in steel, alkali and alkali salts, minerals, and even paint pigments. However, certain constituents of metals such as carbon, sulfur, and phosphorous cannot be determined by this instrument. The cost of arc and spark spectographic equipment is high—$15,000 to $20,000. The additional cost of required accessories increases the total cost for establishing a complete spectrographic laboratory to $40,000 or more. A highway laboratory having a very large volume of metal samples for chemical analysis may be able to justify such equipment.

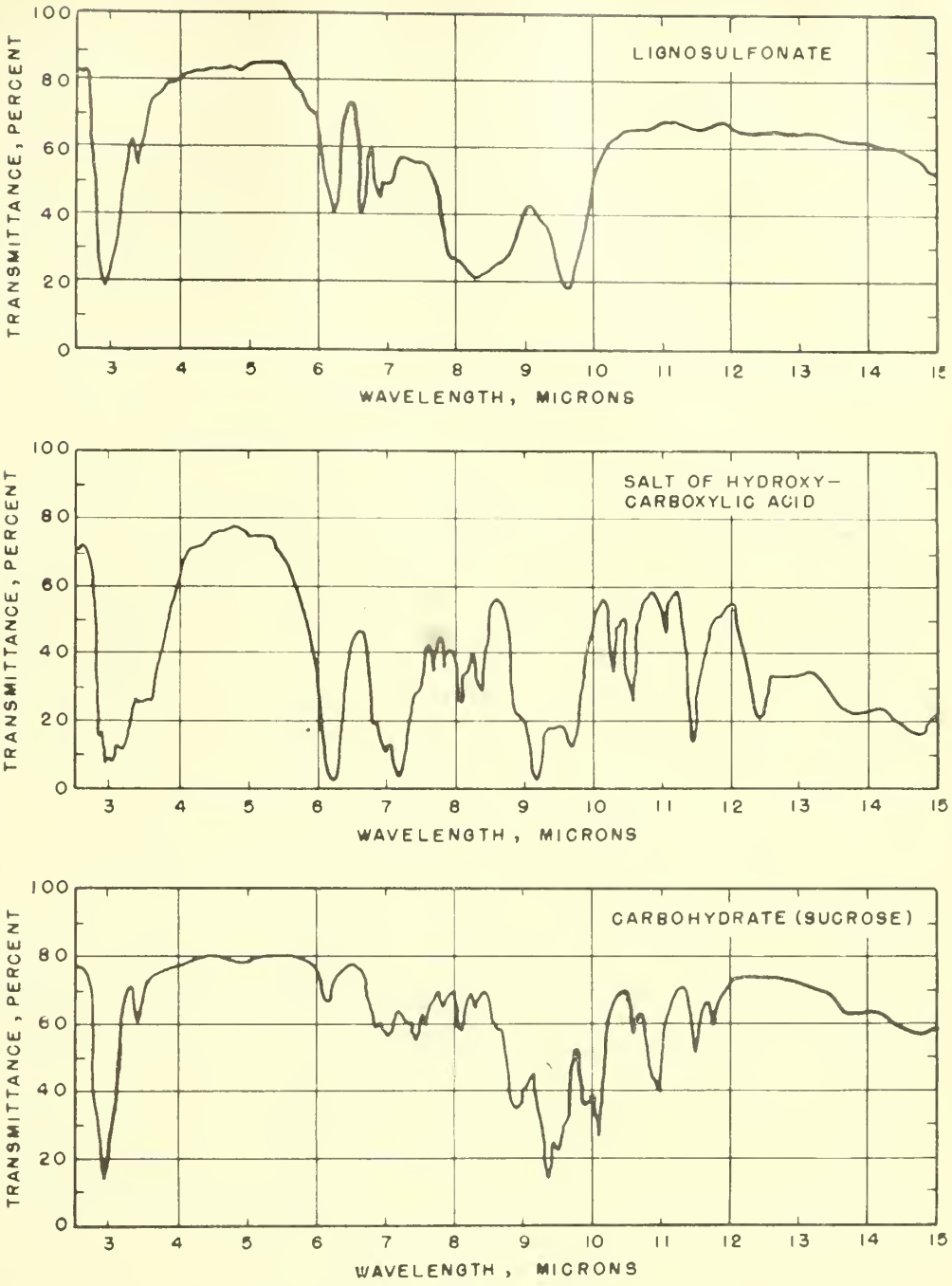

Figure 12.—Infrared spectra of concrete retarding admixtures.

Ultraviolet and Visible Light Spectroscopy

The color of any object is the result of absorption of certain wavelengths of white light and reflection or transmission of others. Thus, the absorption of specific wavelengths of energy by particular substances is a familiar phenomenon even though it may not have been recognized. The technical explanation of the phenomenon is rather involved. It relates to the difference in the energy of the electrons in the sample before and after the absorption of critical radiation. This, in turn, is related to specific characteristics of molecules of substances. The type of curve obtained when the ultraviolet spectrum is recorded is shown in figure 6. For some classes of compounds, the peaks in the curve will always occur in approximately the same region. Thus, qualitatively, the wavelength location of the peak is of some value. However, the greatest application of both ultraviolet and visible light absorption measurements for highway materials is in quantitative analysis. The quantitative measurement is the vertical displacement of the peak—the absorbance of the sample.

It is known that the amount of light absorbed by dilute solutions of a substance is directly related to the amount of dissolved substance present. Thus, the absorption of a test solution is obtained and its concentration is determined from a calibration curve previously established for solutions containing known amounts of the substance under test. A typical calibration curve for a lignosulfonate material is illustrated in figure 7. The same principle is applied to both the ultraviolet and visible light wavelength bands.

Compounds that absorb energy in the ultraviolet region are generally organic in nature and those that absorb energy in the visible region are usually inorganic. The absorbed light in the visible region is related to the color characteristics of the sample.

Application of ultraviolet spectroscopy

At the Public Roads laboratory, ultraviolet spectral analysis is used in conjunction with infrared spectroscopy to control the chemical uniformity of air-entraining and retarding admixtures for concrete. Information on part of this work has been published (*4*), typical ultraviolet spectra of such admixtures for concrete are shown in figure 8. The height of each peak—the absorbance—is directly related to the concentration of the active ingredient.

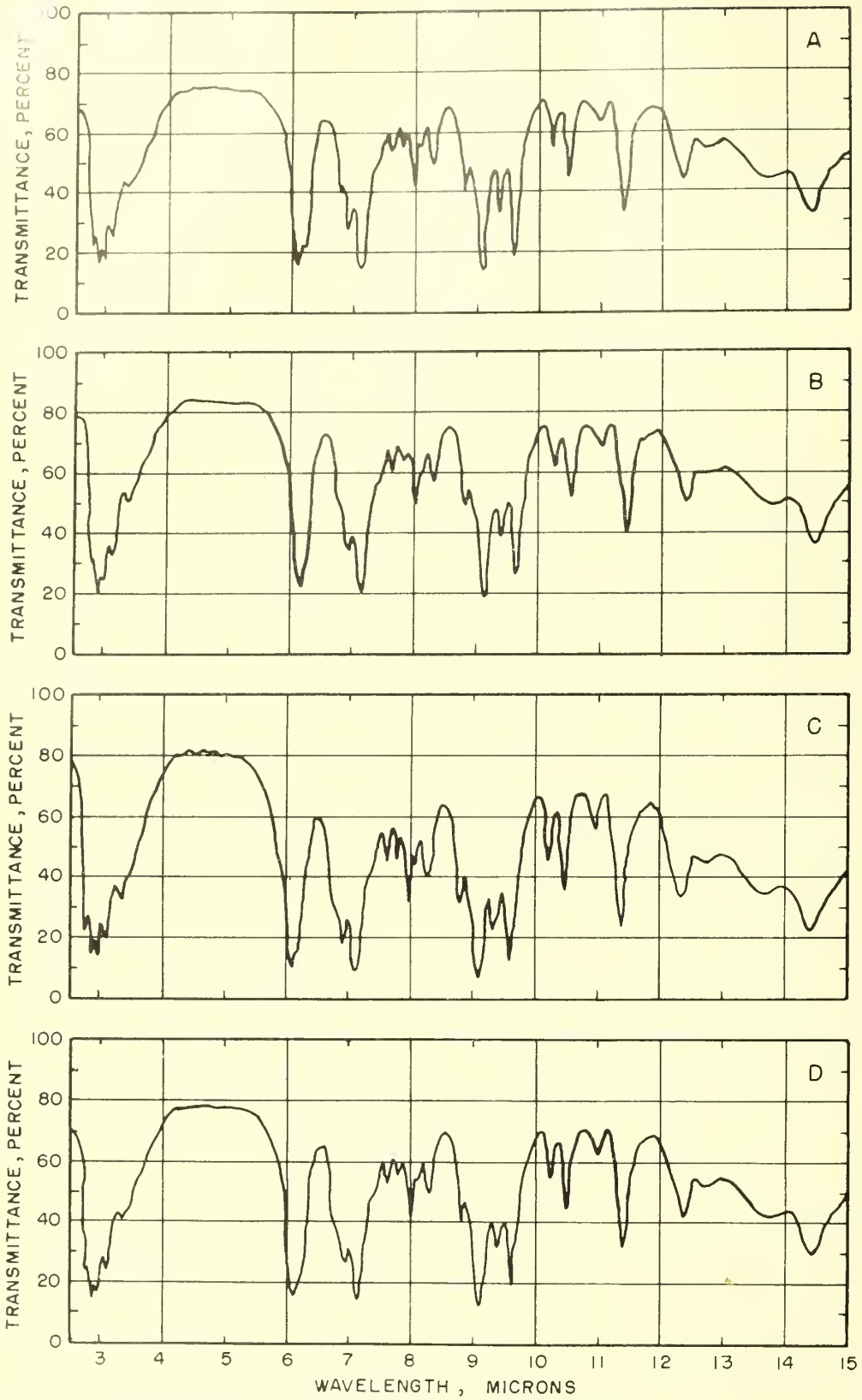

Figure 13.—Infrared spectra of four different lots of the same trade name retarder (hydroxycarboxylic type).

Other laboratories use an ASTM ultraviol spectrophotometric method to determine tl phthalic anhydride content of alkyd resi paints (*5*). The curve for phthalic aci shown in figure 6, has a peak at a wavelengt of 276 millimicrons, which is characteristic this material. The absorbance measuremer (vertical displacement) at 276 millimicrons proportional to the concentration of th phthalic acid in the sample. This relationshi is shown in figure 9.

Application of visible light absorptio spectroscopy

In the visible region of the spectrum, manually operated instrument often is use to determine the presence of constituents suc as phosphorus in fertilizers, titanium in pair pigments and portland cements, and solubl silica in the ASTM chemical test for potenti alkali-aggregate reactivity of concrete aggr gates. Such spectrophotometric methods pr vide much more rapid analyses than ordinar chemical procedures and are especially usefu when the amount of the constituent to b determined is very small.

Instrumentation

Automatic recording ultraviolet and visibl light spectrophotometers are very expensive— from $5,000 to $10,000. These instrument are useful mainly for research and speci investigations on organic materials. Automa tic recording instruments can scan the entir ultraviolet spectrum in a period of time a short as 1 or 2 minutes, whereas the use manually operated instruments for thi purpose may require 1 or 2 hours and consider able effort. For quantitative analyses, bot in the visible and ultraviolet regions, a readin is usually taken at a specific wavelengt consequently, the manually operated instru ments are entirely adequate for such applica tions and their cost is about $2,500, perhap less.

Infrared Spectroscopy

Infrared spectroscopy is similar to ultra violet spectroscopy in that it detects th presence of groups of atoms rather than th element itself, but it differs in that vibratin atoms or groups of atoms, rather than elec trons, are responsible for the absorption of th incident energy. Such atomic groups vibrat with definite, sharply defined frequencies tha are characteristic of the molecular combina tions. These frequencies lie in the infrared o radiant heat region of the electromagneti spectrum, which extends from the red end o the visible region (about 0.8 microns) to 3 microns and beyond. When a sample i placed in a beam of such radiation, it absorb infrared energy at frequencies characteristic o

ne molecule and transmits all other frequencies. The resultant pattern, known as an nfrared absorption spectrum, is probably the ost characteristic physical property of all ompounds. No two compounds of different ructure or different chemical nature will have lentical infrared spectra. For this reason the nfrared spectrum of a material is often re-rred to as a characteristic fingerprint.

Most functional groups (parts of chemical ompounds) have a characteristic absorption t fixed frequencies. Thus, the infrared pectrum of a substance can also indicate the resence or absence in a compound of chemical roups such as carbonyl, hydroxyl, carboxylic cid, ether, vinyl, aromatic rings—just to ention a very few. Consequently, both the riginal chemical nature and structure, as well s the chemical changes occurring during ging or exposure of materials, can be detected y infrared analysis. The intensity of an nfrared absorption peak at a specific wave-ngth is a definite and reproducible function f the amount of material in the sample beam. Hence, the absorption intensity is related to he amount of a given material in the sample.

pplications of infrared spectroscopy

An infrared spectrophotometer in operation s shown in figure 10. The Public Roads aboratory has used infrared analysis for many ighway materials. One such application has been the identification of commercial concrete etarders. Because such materials are sold nder different proprietory trade names, it becomes important for the purchaser to assure himself that the product is uniform in composition from batch to batch. This can be conveniently and rapidly done by infrared nalysis, and long-term or involved physical esting of each batch of product purchased is ot necessary.

Infrared spectroscopy provides an ideal and apid tool, as well as one of the surest methods, or the analysis of organic substances of all ypes. With some limitations, it is also applicable to the analysis of inorganic materials. The sample can be in the form of a solid, iquid, or gas, and the complete analysis of a material is extremely rapid. Generally, quantitative determinations based on the amount of absorption are accurate to within plus or minus 5 to 10 percent of the amount of the constituent present.

Sample size can be very small—as little as 1 mg., less if necessary. Very little sample preparation is required, except that water or moisture must first be removed from the sample. Only minutes are required for a complete recording of the spectrum. Some States have used infrared spectral analysis for the identification of paint vehicles. Modern-day paint resins are so complex that they almost defy identification by use of ordinary chemical methods. Therefore, infrared has become a valuable method for use in checking supplies of traffic paints against the original paint supplied for the performance tests. Significant differences in composition can be clearly demonstrated in minutes by infrared analysis; whereas, the detection of such differences by chemical means would require days—if they could be detected at all. Figure 11 shows the spectra of different paint resins used in traffic paints.

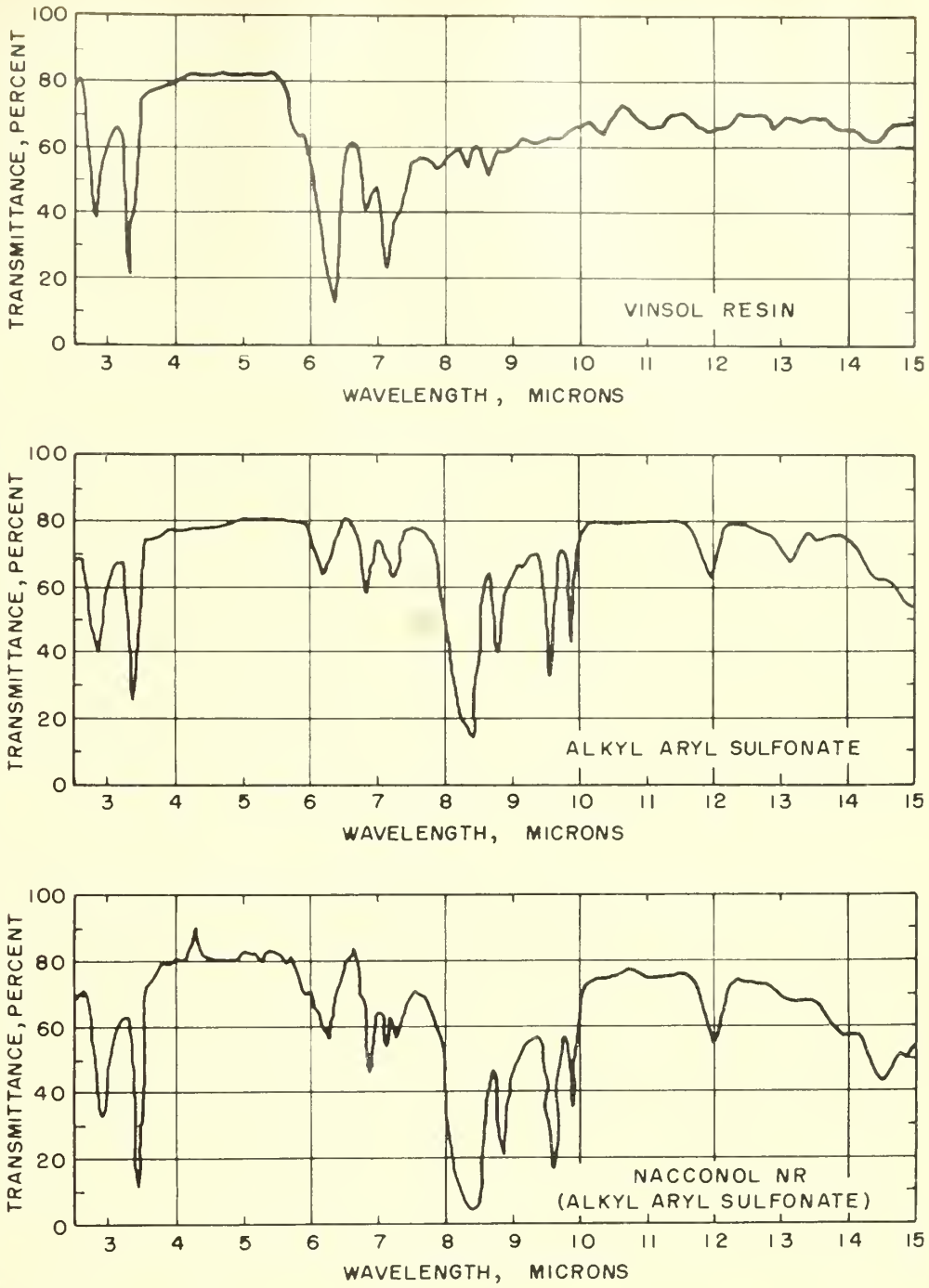

Figure 14.—Infrared spectra of concrete air-entraining admixtures.

Infrared is also applied in a similar way to organic materials that are used as concrete admixtures. The infrared patterns of several different classes of retarders are shown in figure 12. Notice that the three spectra are substantially different and that each is characteristic for that proprietory product. Figure 13 shows how infrared can be used as a check on product uniformity. Each spectrum represents a different lot of the same pro-

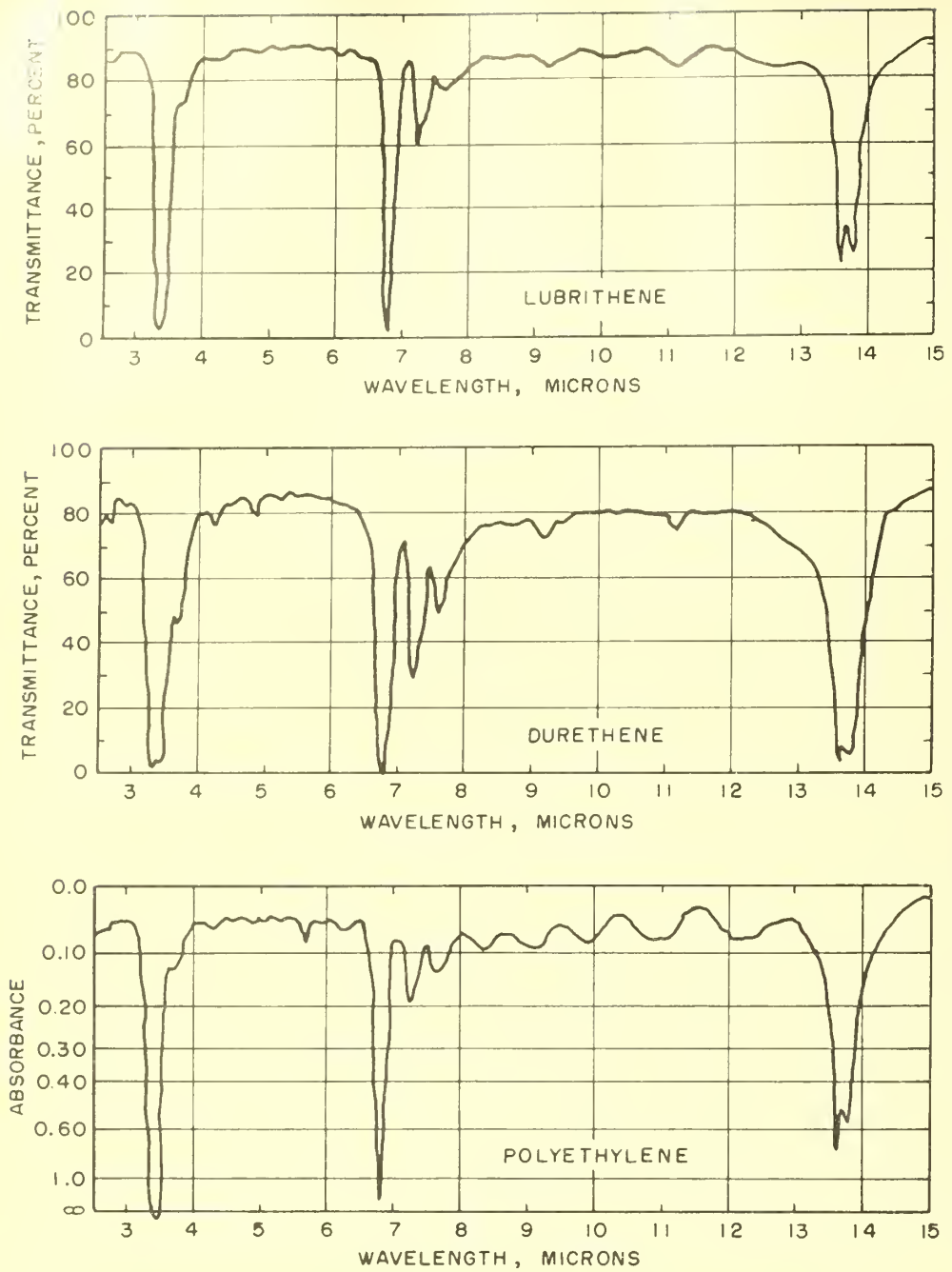

Figure 15.—Infrared spectra of plastic curing materials for concrete.

prietory product. The position of the peaks as well as the shapes of the curves are substantially the same, thus giving assurance that the product is uniform.

The list of both organic and inorganic materials that can be analyzed by infrared spectroscopy is almost endless and many of these materials are of interest to highway laboratories. In addition to those already mentioned, present-day practical applications may be made to materials such as air-entraining agents, herbicides, epoxy resins, silicones, plastic sheeting and membrane curing materials, synthetic rubber water-stops, clays, and other minerals. The infrared patterns of some of these materials are shown in figs. 14–17.

The infrared spectrophotometer is also a valuable tool in the continuing research on the nature and composition of asphalts. Typical infrared spectra of asphaltic constituents, separated from asphalt by chromatographic methods, are shown in figure 18.

Instrumentation

Infrared instruments range in price from $5,000 to $25,000. A model in the lower price range is generally considered adequate for the purposes discussed. Infrared equipment can be economically justified where concern is either with research problems and the chemical nature of organic materials or the control of chemical uniformity of organic trade products.

X-ray Diffraction

X-ray diffraction is used to analyze crystalline materials in the solid state. When the crystalline material is irradiated with a monochromatic beam of X-rays, the different planes of atoms within the sample will diffract the X-ray beam at angles determined by the spacing between the planes. These diffracted beams can be recorded on a film placed concentrically around the sample, and the resultant image is called an X-ray diffraction pattern. This pattern is unique for each material irradiated and varies according to the characteristics. The instrument is usually further refined to provide quantitative results by employing a counter and the results obtained are in the form of patterns similar to that for X-ray fluorescence shown in figure 19. The location or lateral displacement of the peaks qualitatively establishes the nature of the material, and the height of the peaks is used as a quantitative measure of the crystalline constituents.

In operation, a small powdered sample is inserted in the instrument and the quantitative X-ray diffraction pattern is recorded automatically within 30 minutes. No special preparation normally is required for ordinary solid samples.

Application and instrumentation

X-ray diffraction techniques are applicable to studies of highway materials such as soils, clays, and aggregates. By use of this technique, the mineralogical nature of the material may be evaluated. X-ray diffraction may be used in soils and soil stabilization studies to evaluate the clay mineral nature of the soil and thereby assist in evaluation of the potential soil or subgrade properties. Siliceous minerals such as quartz and tridymite or calcareous minerals such as calcite and dolomite may be distinguished from each other, as well as identified in aggregates; this identification can be helpful when knowledge of the mineral nature of the aggregate is important. Each mineral present can be quantitatively estimated to the nearest 5 percent of the sample.

The cost of X-ray diffraction instrumentation is approximately $25,000. At the present ime, these instruments are used by highway nd university laboratories in connection with esearch investigations of portland cement, oils problems, or other special investigations f a nonroutine nature as to the mineralogical ature of highway materials. Unless a laboatory is concerned with such problems regularly, the cost of the instrument probably ould not be justified.

X-ray Fluorescence

The principle of X-ray fluorescence is rather nvolved. It is based on changes in the energy evels of the electrons of the atoms that cause X-ray radiation or fluorescence. These fluoescent X-ray photons, when identified by vavelength measurements, characterize the element, and their frequency of occurrence is quantitatively proportional to the amount of he element present. The type of spectrum obtained is similar to those obtained by other X-ray techniques and is illustrated in figure 19.

X-ray fluorescence is suitable for analyzing either liquid or solid samples and is useful or the identification and quantitative determination of the elements that are present in an inorganic material. At the present time, application of this technique is limited to elements having an atomic number of 12 or more—12 is the number for magnesium. Therefore, analyses for elements such as sodium, oxygen, nitrogen, carbon, lithium, hydrogen, and a few others cannot now be made.

Quantitative accuracy of results obtained by the X-ray fluorescence technique is about 1 percent of the constituent present. Accuracy is even better when the approximate amounts of the other elements present in the sample are known so that suitable standards can be used for calibration. Use of X-ray fluorescence is not limited to the quantitative analysis of only the minor constituents of a material as is arc and spark spectroscopy; its greatest utility is and will be in determining the quantitative amounts of major elemental constituents present in a material. Sample preparation requires about 15 minutes, and a quantitative determination for each element also requires approximately 15 minutes. It takes about 1 hour to qualitatively scan the entire spectrum for the presence of all possible elements, but routine analysis of specific materials can be conducted in approximately 30 minutes.

Application and instrumentation

Examples of materials that can be analyzed at the present time by X-ray fluorescence include the chemical elements in steel, alloys, and paint pigments. Research studies are now being conducted by the National Bureau of Standards and other organizations to determine its usefulness in the analysis of portland cement. At the present time, its

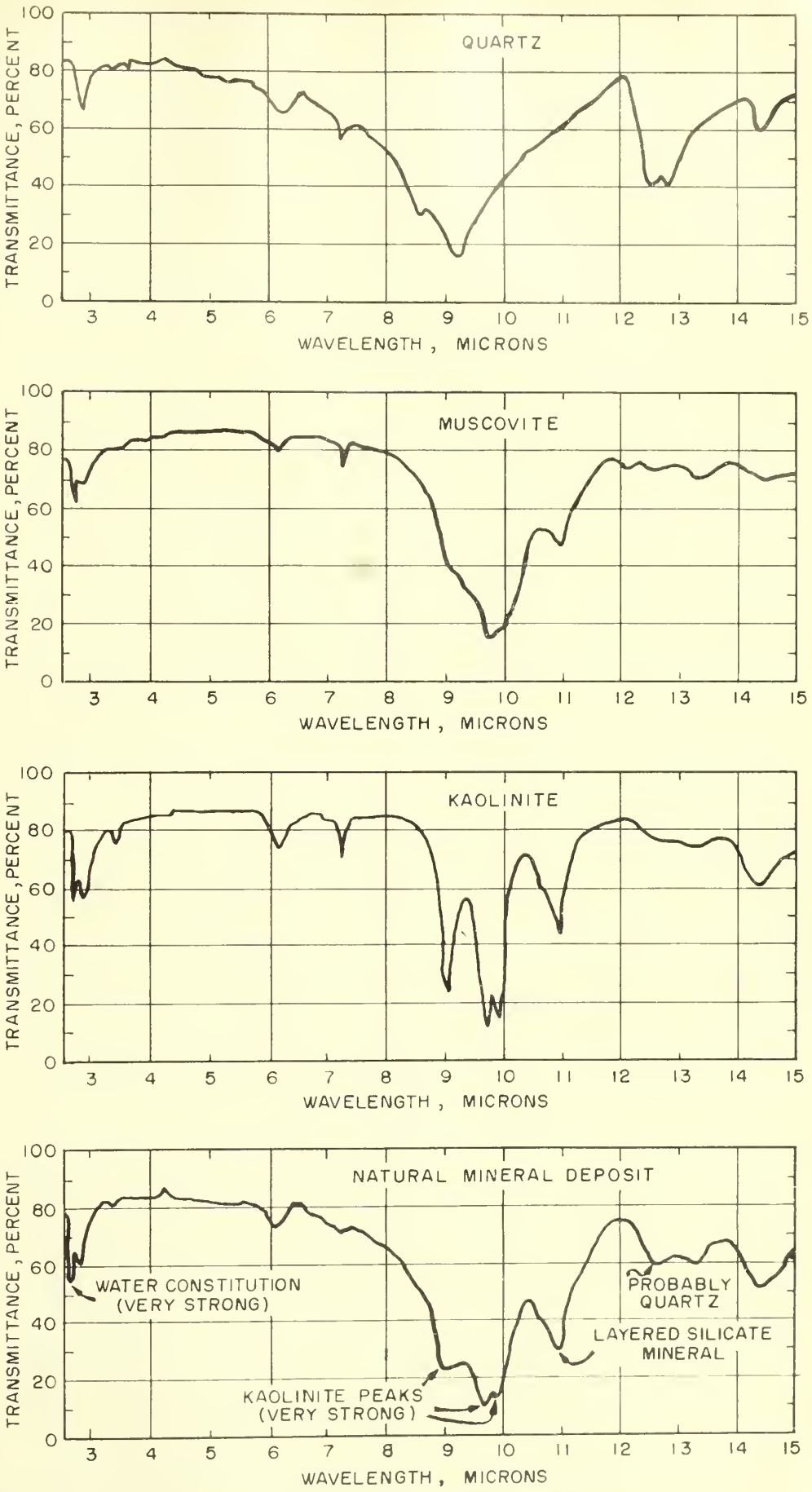

Figure 16.—Infrared spectra of clays and minerals.

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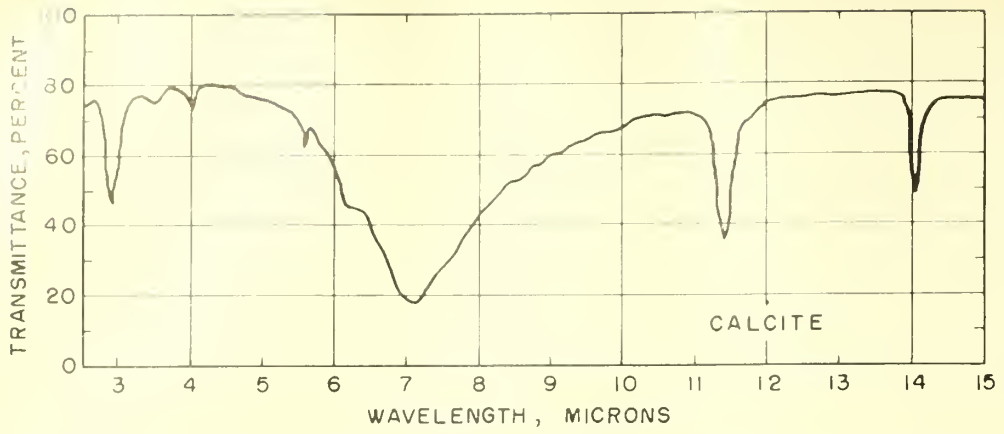

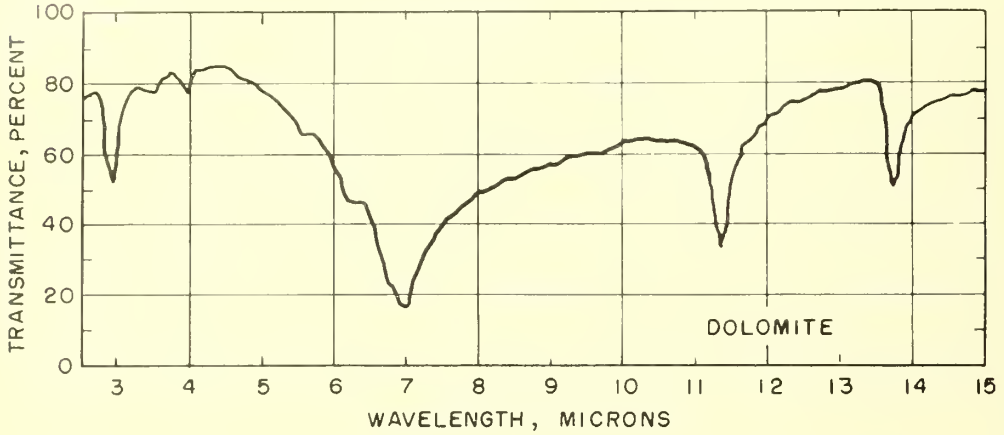

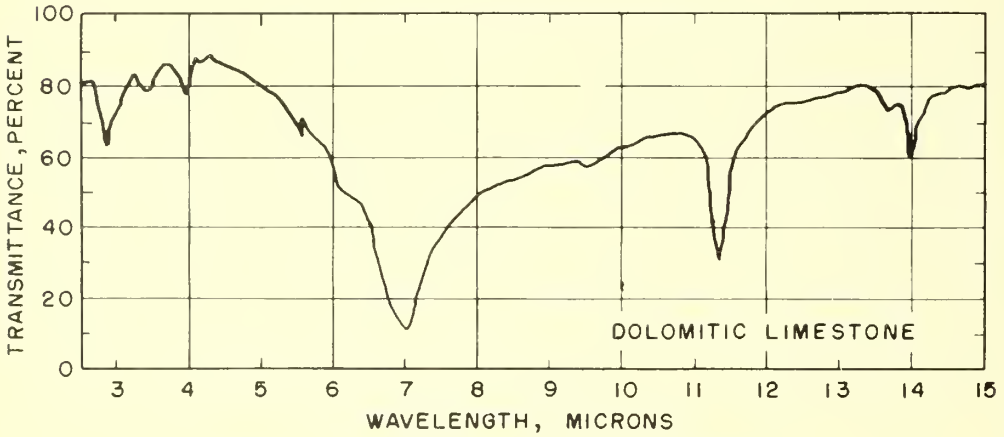

Figure 17.—Infrared spectra of carbonate minerals.

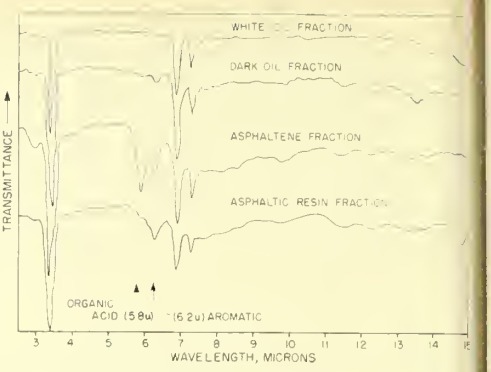

Figure 18.—Infrared spectra of component of asphalt.

application to this material is not considered practical for rigid control purposes but such application is expected to become a reality in a little more than 5 years. This would mean that an analysis now requiring several days could be completed in from 30 minutes to an hour.

The instrument is usually set up for the analysis of a particular material—a specific alloy, portland cement, or special steel. Consequently, in its present stage of development, it is impracticable to adjust the same instrument for use on a cement, then a paint pigment, and then an alloy, etc. Therefore, the future use for X-ray fluorescence appears most promising for the analysis of large volumes of specific products that have a rather uniform nature. The cost of such instruments ranges from a minimum of $5,000 to more than $45,000, depending upon the channels or selectors provided for the different elements.

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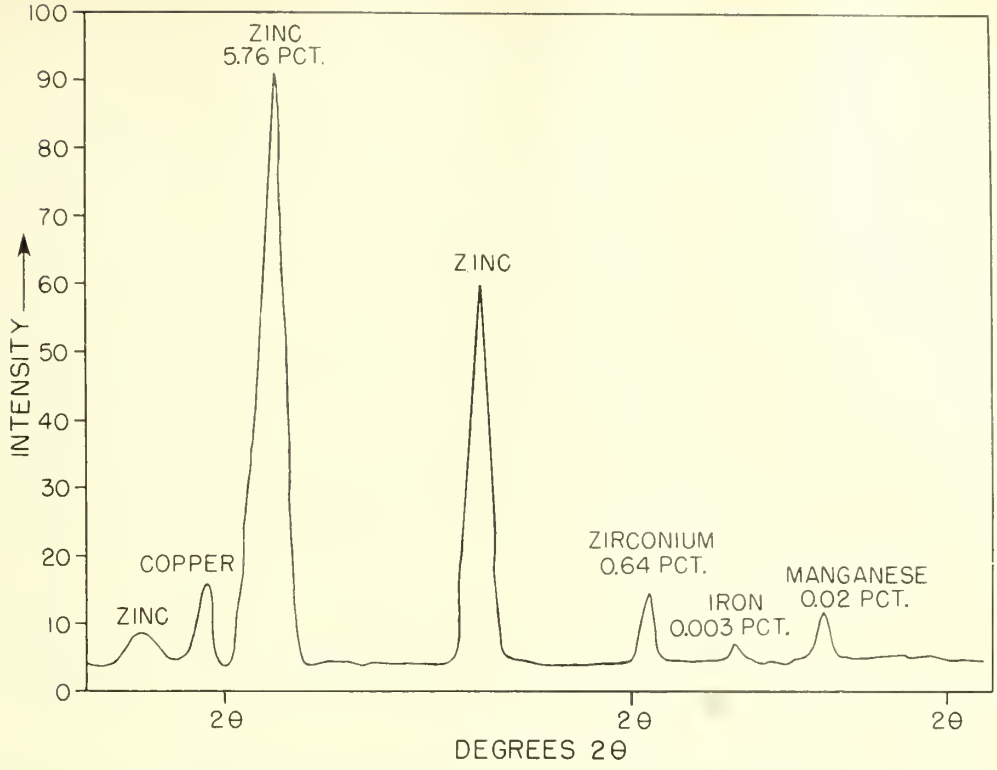

Figure 19.—X-ray fluorescence spectrum of magnesium alloy.

Table 1.—Spectroscopy methods presently applicable to routine control of highway materials [1]

Type of instrument	Approximate cost including accessories	Fields of application: Material	Fields of application: Constituent determined and/or purpose of test	Fields of application: Type of determination	Average attended time required per sample [4]: For sample preparation	Average attended time required per sample [4]: For instrumental evaluation and calculation	Average attended time required per sample [4]: Total	Average elapsed time for analysis [7]	Quantitative accuracy; percent of constituent present: Routine	Quantitative accuracy; percent of constituent present: Possible
					Minutes	*Minutes*	*Minutes*	*Days*	*±Percent*	*±Percent*
Spectrophotometer: (manual type)	$2,500									
Ultraviolet light	[2]	Paint vehicles	Alkyd resin	Quantitative	5–30	10	15–40	1	2	0.5
		Concrete retarders	Lignosulfonate	do	5–30	10	15–40	1	2	0.5
		Concrete air-entraining admixtures.	Vinsol resin, Darex, etc.	do	5–30	10	15–40	1	2	0.5
Visible light		Fertilizer	Phosphoric acid	do	5–30	5	10–35	[5] 1	2	0.5
		Cement, slag	Titanium dioxide	do	5–30	5	10–35	[5] 1	2	0.5
		Paint pigments	do	do	5–30	5	10–35	[5] 1	2	0.5
		Concrete aggregates	Alkali reactivity, chemical test for silica.	do	5–30	1	10–35	[5] 1	2	0.5
		Steel	Minor constituents	do	[5] 5–30	[5] 5	[5] 10–35	[5] 1	2	0.5
		Copper & aluminum alloys	do	do	[5] 5–30	[5] 5	[5] 10–35	[5] 1	2	0.5
Flame Photometer	[3] $500–2,500	Cement	Alkalies	do	5–15	5	10–20	½	5	1.0
		Fertilizer	Potash	do	5–15	5	10–20	½	5	1.0
		Rock salt (for ice removal)	Sodium chloride	do	5–15	5	10–20	½	5	1.0
Infrared spectrophotometer. (automatic recording)	$5,000–25,000	Concrete retarders, water-reducers, air-entraining admixtures, curing materials, and rubber and synthetic water stops.	Identification and/or uniformity of product.	Qualitative and semi-quantitative	5–20	15–30	20–50	1	10	2.0
		Herbicides	do	do	5–20	15–30	20–50	1	10	2.0
		Epoxy resins	do	do	5–20	15–30	20–50	1	10	2.0
		Paint vehicles and pigments	do	do	5–20	15–30	20–50	1	10	2.0
		Silicones	do	do	5–20	15–30	20–50	1	10	2.0
		Carbonate aggregates	do	do	5–20	15–30	20–50	1	10	2.0
		Clays and other minerals	do	do	5–20	15–30	20–50	1	10	2.0
		Bituminous mineral fillers	do	do	5–20	15–30	20–50	1	10	2.0
Arc and spark spectrograph. (Photographic plate)	$40,000	Alloys of aluminum, copper, magnesium, nickel, lead, tin, and zinc.	Minor alloying constituents.	Quantitative	[6] 0–10	[6] 5–50	[6] 5–60	[6] 1–3	10	1.0
		Steel	do	do	[6] 0–10	[6] 5–50	[6] 5–60	[6] 1–3	10	1.0
		Paint pigments	Minor constituents	do	[6] 0–10	[6] 5–50	[6] 5–60	[6] 1–3	10	1.0

[1] Date shown in table are general estimates. Considerable variation in estimates should be expected depending on specific conditions encountered.
[2] Available as accessory to manually operated spectrophotometer and is included in cost shown for this instrument.
[3] Also available as accessory to some manual spectrophotometers.
[4] Assuming large volume of similar samples to be tested.
[5] Time shown is for each alloying constituent in metals.
[6] Time shown is for all of the alloying or minor constituents present.
[7] Time from beginning of work on a specific sample until analysis is complete for that sample under routine control conditions.

New Publications

INTERSTATE SYSTEM ROUTE LOG AND FINDER LIST

The Bureau of Public Roads has recently published *Interstate System Route Log and Finder List,* a 16-page leaflet that explains the numbering system of the National System of Interstate and Defense Highways and presents (1) a list of main Interstate Highway System routes, the mileage in each State, and key cities the routes pass through; (2) a list of radial, circumferential, and spur Interstate routes; and (3) a list of major cities served by the Interstate System. A small-scale map of the System is also included.

These listings provide a means for finding the general locations of each Interstate route and for finding the numbers of the routes that serve each major city. Because many portions of the Interstate System are not yet built, the leaflet will not serve as a touring guide, nor is it intended for that purpose. It will be useful, however, for many other purposes.

This leaflet is available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., for 10 cents a copy.

HIGHWAY STATISTICS, 1961

The Bureau of Public Roads, U.S. Department of Commerce, has published a new 150-page bulletin, *Highway Statistics, 196*. the 17th in the annual series that presen statistical and analytical tables of genera interest on motor fuel, motor vehicles, higl way-user taxation, State and local highwa financing, road and street mileage, and Fe eral aid for highways.

Highway Statistics, 1961, may be purchas from the Superintendent of Documents, U.S Government Printing Office, Washington 2 D.C., for $1.00 a copy. Some of the previou annual issues of the series and the summar to 1955 are also available from the Superin tendent of Documents; a list of availabl issues is carried on the inside back cover o this magazine.

The Road to Better Roads

(Third AASHO Road Test Film)

A third film on the AASHO Road Test has been released by the Bureau of Public Roads, U.S. Department of Commerce. Entitled, *The Road to Better Roads,* this film is a 16-mm. color and sound production that has a running time of about 14 minutes. This film is a nontechnical description of the $27 million highway research project conducted at Ottawa, Illinois during the years 1956 to 1961.

The Bureau of Public Roads has produced two previous films designed to acquaint engineers with the technical details of the project's materials, construction, test procedures, and results. Information on these films has been published in PUBLIC ROADS, volume 32, No. 3, p. 63, and No. 5, p. 112, respectively.

The latest film is considered suitable for showing to citizens groups, highway-oriented organizations, or legislative bodies. A Public Roads spokesman has suggested that the film might be particularly useful to State highway officials in explaining the background of State-conducted research designed to expand upon the results of the AASHO Road Test.

The Road Test was a research project designed to study the performance of different highway pavements and bridges under controlled truck and traffic loading. The test was sponsored by the American Association of State Highway Officials and financed by the States, Public Roads, and some industry groups. The Department of Defense cooperated in the project, which was administered by the Highway Research Board of th National Academy of Sciences—Nationa Research Council.

Prints of the film, *The Road to Better Roads* may be borrowed by contacting one of th Division Offices of the Bureau of Publi Roads, one of which is located in each Stat capital. Requests may also be submitte directly to the Photographic Section, Bureau of Public Roads, Washington 25, D.C. There is no charge other than for express or postage fees for booking the film. Requests should be submitted well in advance of the desired screening date, and alternate dates indicated if possible. Immediate return of the film after each booking is required.

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EPORTS TO CONGRESS

actual Discussion of Motortruck Operation, Regulation and Taxation (1951). 30 cents.

ederal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

lighway Cost Allocation Study:

First Progress Report, House Document No. 106 (1957). 35 cents.

Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.

Final Report, Part VI: Economic and Social Effects of Highway Improvement, House Document No. 72 (1961). 25 cents.

he 1961 Interstate System Cost Estimate, House Document No. 49 (1961). 20 cents.

J.S. HIGHWAY MAP

Iap of U.S. showing routes of National System of Interstate and Defense Highways, Federal-aid Primary Highway System, and U.S. Numbered Highway System. Scale 1 inch equals 80 miles. 25 cents.

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Highway Bond Calculations (1936). 10 cents.

Highway Capacity Manual (1950). $1.00.

Highway Statistics (published annually since 1945):
1955, $1.00. 1956, $1.00. 1957, $1.25. 1958, $1.00. 1959 $1.00. 1960, $1.25. 1961, $1.00.

Highway Statistics, Summary to 1955. $1.00.

Highway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.

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Parking Guide for Cities (1956). Out of print.

Peak Rates of Runoff From Small Watersheds (1961). 30 cents

Road-User and Property Taxes on Selected Motor Vehicles, 1960. 30 cents.

Selected Bibliography on Highway Finance (1951). 60 cents.

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Vol. I—Concrete Superstructures. $1.00.
Vol II—Structural Steel Superstructures. $1.00.
Vol. III—Timber Bridges. $1.00
Vol. IV—Typical Continuous Bridges. $1.00.

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The Role of Aerial Surveys in Highway Engineering (1960). 40 cents.

Transition Curves for Highways (1940). $1.75.

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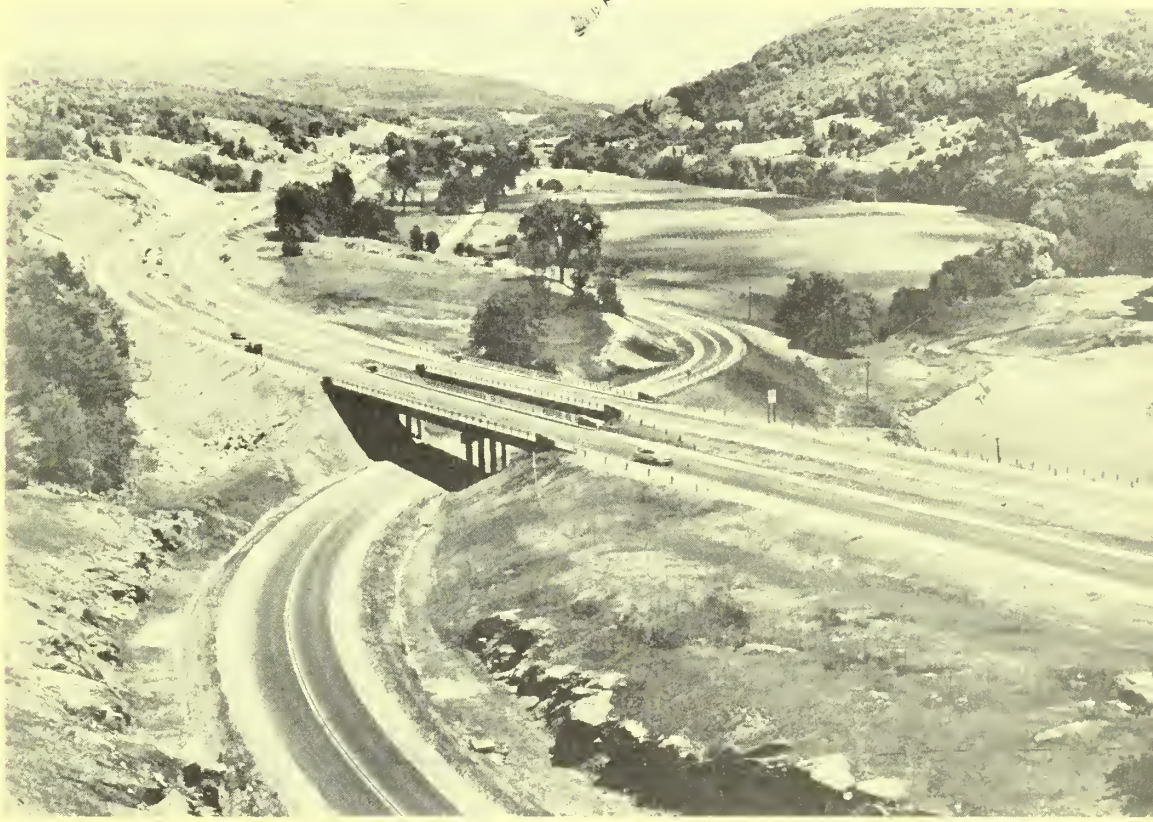

Interstate Highway 89 west of Waterbury, Vermont.
This view faces east up the Winooski River Valley; in the center of the photograph I–89 overpasses relocated US–2 on twin bridges.

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Errata

In vol. 32, No. 7, April 1963, PUBLIC ROADS change IN THIS ISSUE the initials of Mr. Shufflebarger to read: C. L. and in the title, *Estimated Travel by Motor Vehicles in 1962*, change the date to *1961*.

U.S. DEPARTMENT OF COMMERCE
LUTHER H. HODGES, Secretary

BUREAU OF PUBLIC ROADS
REX M. WHITTON, Administrator

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 6, 1961.

Contents of this publication may be reprinted. Mention of source is requested.

Analyses of Direct Costs and Frequencies of Illinois Motor-Vehicle Accidents, 1958

By[1] **CHARLES M. BILLINGSLEY, Transportation Economist,**
THE U.S. BUREAU OF PUBLIC ROADS, and
DAYTON P. JORGENSON, Research Analyst,
ILLINOIS DIVISION OF HIGHWAYS

The Illinois Accident Cost Study was designed to provide comprehensive data on the cost of motor-vehicle accidents and incidents of all degrees of severity, ranging from incidents involving only a few dollars damage to the most severe and costly accidents.

During the study year nearly 10 million persons resided in the State and more than 4 million of these individuals were licensed to drive. Also, nearly 3⅓ million privately owned Illinois passenger cars and trucks were in use during 1958, and one-tenth of the owners of these vehicles were involved in an accident of such severity as to require an owner's report to be filed with the Bureau of Traffic, Division of Highways.

Data developed in the Illinois study, as well as in earlier studies conducted in other States, indicated that a substantial part of the accident problem is overlooked by basing studies only upon the official accident records of a State. Findings of the Illinois study showed that approximately three-fourths of the total number of accident involvements and slightly more than two-fifths of the total direct costs were attributed to unreported events. Although most of the unreported involvements were minor happenings, in the aggregate they represented a significant part of the total direct cost of motor-vehicle accidents in Illinois.

Motor-vehicle accidents generate a wide array of tangible and intangible effects. A satisfactory appraisal of all the economic and social consequences of an accident is a practical impossibility in statistical terms. Thus, to interpret the data presented in this article, it is necessary to emphasize that only the costs directly associated with accidents were included.

Although the study was restricted to the determination of direct costs, the task of accounting for and verifying all cost items accruing from a vehicle being involved in an accident was considerable. Interviewers traveling over the 102 counties of the State made nearly 23,000 personal and 4,000 telephone calls. To ferret out and verify costs, they completed 16,600 interviews with owners and drivers of vehicles, with injured persons and their relatives, with doctors, insurance brokers, attorneys, police officials, and others.

Introduction

SOME OF the principal findings of the Illinois motor-vehicle accident cost study, a cooperative project of the Illinois Division of Highways and the U.S. Bureau of Public Roads, are discussed in this article. The study, which was undertaken in 1959, was designed to measure the direct costs of accidents and incidents involving owners of Illinois registered passenger cars and trucks during calendar year 1958 and to relate such costs to the highway, the vehicle, and the persons involved.

The only distinction between a motor-vehicle accident and a motor-vehicle incident is the element of motion. In an incident, there is no motion on the part of the motor vehicle. In general, losses through motor-vehicle incidents include such events as storm damage, acts of vandalism, fires, mishaps occurring during the servicing and repair of a motor vehicle, collisions of conveyances other than motor vehicles with parked or standing motor vehicles, and similar happenings.

Many cost items can be associated with traffic accidents and other mishaps, but cooperative studies of the Bureau of Public Roads and State highway organizations undertaken to date have been concerned only with the direct costs of accidents and incidents.[2] A broad but not quite accurate definition is that the Illinois study and previous studies have reflected only the "out-of-pocket" costs. Stated more precisely, the costs were those directly attributable to accidents, and the costs thus determined represented the use of resources that would have been available for other purposes had the accidents not occurred. Cost elements included in the study are discussed in subsequent sections of this article.

Legal requirements in all States specify that owners must file a report of motor-vehicle accidents involving death or injury. The laws relating to property damage only accidents vary from State to State. In Illinois the statutory requirement specifies that an accident report must be filed with the State for any motor-vehicle accident involving death or injury, and any accident in which damage to property of any one person exceeds $100.

With few exceptions, accident statistics published by different private and public organizations are based solely upon official reports of accidents filed with State agencies. Data developed in cooperative studies undertaken by the Bureau of Public Roads have indicated that a substantial part of the accident problem is overlooked by relying only upon official accident records. Many accidents occur for which no reports are filed, and although the events are usually minor happenings they add significantly to the number and cost of accidents.

The Illinois study, as well as previous studies, was designed to determine the direct costs of accidents and incidents ranging from minor fender-denting collisions to the most serious accidents involving death or injury.

---

[1] Presented at the 42d annual meeting of the Highway Research Board, Washington, D.C., January 1963.

[2] Other cooperative studies and the year of survey were: Passenger car phase—Massachusetts, 1953; New Mexico, 1955; and Utah, 1955. Truck phase—Massachusetts, 1955; New Mexico, 1956; and Utah, 1957.

Summary

The major findings of the Illinois accident cost study discussed in this article are presented in the following paragraphs.

Direct costs of motor-vehicle accidents and incidents involving Illinois registered passenger cars during 1958 totaled $309.5 million. For Illinois trucks, such costs amounted to $29.3 million. For events, occurring both on and off the highways and in and out of the State of Illinois, the resultant costs to persons and property totaled one-third of a billion dollars or an amount equivalent to three-fifths of the total outlay of funds by State, Federal, and local governments for the construction and maintenance of Illinois roads and streets during 1958.

The one-third of a billion dollars represented an average cost of $928,000 per day, $104 per vehicle in use, $84 for each person having a permit to drive, and $35 per capita.

Approximately 1.3 million Illinois passenger cars were involved in traffic accidents on Illinois highways costing $258.8 million, or an average of $196 per event; similarly, 128,000 trucks were involved in traffic accidents costing $18.1 million, or an average of $141 per event. A further comparison on the basis of exposure indicated costs of 0.97 of a cent per passenger-car mile and 0.36 of a cent per truck-mile.

Three-fourths of the 1.3 million passenger car involvements and four-fifths of the 128,000 truck involvements were not recorded in the official accident files of the State. Although most of these events were minor happenings in which property damage costs were below the legal reporting minimum, they accounted for 42 percent of the total direct costs of passenger car accidents and 55 percent of the total direct costs of truck accidents.

The distribution of the accident cost dollar for all severity classes of accidents was, as follows: Property damage, 60 cents; treatment of injuries, 8 cents; loss of use of vehicle, 1 cent; value of work time lost, 8 cents; legal and court fees, 10 cents; and damage awards and settlements in excess of known costs, 13 cents.

The problems inherent in sampling the universe of traffic accidents for the purpose of determining costs were made evident by the wide range in costs found for the different severity classes of accidents. Extreme cost values for individual sample cases were, as follows: Fatal injury involvements, $136,000; nonfatal injury, $73,000; and property damage only, $30,000. In contrast, median cost values were $2,280 for fatal injury involvements, $310 for nonfatal injury involvements, and $50 for property damage only involvements.

Passenger car owners were involved in accidents within municipalities 3½ times as often as in rural areas. For truck owners, the ratio was 1 involvement in rural areas for every 5 involvements in municipalities. Costs per passenger-car mile ranged from 0.61 of a cent in rural areas to 1.18 cents in municipalities; similarly, costs per truck-mile ranged from 0.32 of a cent to 0.42 of a cent, respectively.

Comparisons made of accident frequencies and costs by major highway systems indicated that roads and streets of a local character had the least desirable rates. Many of the accidents that took place on residential streets were relatively minor events but, when considered in the aggregate, they represented a sizable part of the total direct costs of traffic accidents.

Sampling Procedure

To attempt a study of Statewide vehicle owners' accident experience for a 1-year period dictated the use of the sampling method. Two sources were used: Owners' accident reports filed with the Illinois Division of Highways, Bureau of Traffic; and registration lists of vehicle owners published by the office of the Illinois Secretary of State. Official accident reports filed with the State during 1958 represented the known population of motor-vehicle accidents. Vehicle owners selected from registration lists represented the unknown area in determining accident and incident occurrence.

The sampling unit used for reported accidents was the license plate number of a privately owned passenger car or truck involved in an accident. Reports on file yielded 320,700 license numbers of Illinois registered passenger cars (or the equivalent of that number) involved in accidents and 26,200 trucks. These data were available on tabulating cards, thus permitting the selection of samples by machine method. The cards were grouped according to severity classes—fatal injury, nonfatal injury, and property damage only—and each group was systematically sampled. Truck involvements were further stratified on the basis of two major vehicle types—single units and truck combinations.

To explore the unknown area of accident and incident occurrence for which no owners' reports were on file with the State, approximately 14,000 license plate numbers, equally divided between passenger cars and trucks, were selected from vehicle registration lists. Passenger car license plate numbers were selected at random and no consideration was given to size or weight of vehicle; truck license plate numbers were stratified on the basis of light, medium, and heavy registered weights and different sampling rates were applied thereto. In Illinois a license plate remains with the owner and may be transferred to another vehicle in the event a vehicle is replaced. The 14,000 vehicle owners thus selected were requested to enumerate their *total* accident and incident experience for 1958 involving the vehicle or vehicles bearing the designated license plate number.

Obviously, as owners selected from vehicle registration lists were requested to give total accident and incident experience, such events reported by owners had to be checked against the official accident records of the State to eliminate happenings that had a chance of being selected in samples of officially reported accidents. Accordingly, those events reported by owners in response to the mailed questionnaire for which a record could be found in the State's files were dropped from the study. The remaining unmatched groups of ac dents and incidents were processed as un ported events. Details concerning sampli procedures, rates of return, data collecti and processing methods have been descrit at considerable length in a previous rep and need not be repeated here (*1*).[3] In t aggregate the study produced 7,184 sam cases of passenger cars and trucks involved an accident or incident.

Frequent mention is made throughout tl article of the cost of passenger car accidents opposed to the cost of truck accidents. A though the passenger car and truck phases the study were conducted concurrently, th were in effect two separate surveys. This a proach was used because the two classes vehicles represented different universes, n only from the standpoint of numbers of vel cles registered and frequencies of acciden but also from the consideration of vehicle a vehicle-use characteristics.

Definitions

In general, the terms used throughout tl study conform with the definitions given the manual, *Uniform Definitions of Mot Vehicle Accidents*, adopted by the Nation Conference on Uniform Traffic Accident St tistics. To aid the reader, some of the co monly used terms are defined here.

Motor-vehicle traffic accident.—Any accide occurring on a trafficway (street, road, hig way), causing death, injury, or property dan age that involves a motor vehicle *in motion* a motor-vehicle traffic accident.

Motor-vehicle nontraffic accident.—Any acc dent involving a motor vehicle *in motion* tha occurs entirely on private property or in an place other than a trafficway and causes deat injury, or property damage is a motor-vehic nontraffic accident.

Motor-vehicle traffic incident.—Any incide involving a motor vehicle *not in motion* tha occurs on a trafficway and causes deat injury, or property damage is a motor-vehic incident.

Motor-vehicle nontraffic incident.—Any inc dent involving a motor vehicle *not in motio* that occurs entirely on private property or any other place that is not a trafficway an causes death, injury, or property damage is motor-vehicle nontraffic incident.

Involvement.—An involvement is defined a a vehicle involved in an accident. As th sampling unit for the study was a license plat number of a vehicle involved in an accident the cost data developed were the accumulatio of costs surrounding selected vehicles involve in accidents and/or incidents. The costs thu determined were factored on the basis of sam ple selection rates and appropriate adjust ments were made for incompleted cases. Th term involvement is a useful expression in de scribing the components of an accident, tha is, size and weight of vehicle involved, age o vehicle, age and sex of driver, etc.

Accidents as such were not sampled in th study because of the procedural difficulties in

[3] References indicated by italic numbers in parenthese are listed on page 213.

Table 1.—Direct cost of accidents and incidents in Illinois, involving Illinois registered passenger cars and trucks

Accident or incident class	Direct cost of accidents and incidents involving—	
	Passenger cars	Trucks
	1,000 dollars	*1,000 dollars*
Traffic accidents	258,770	18,081
Nontraffic accidents	8,514	1,951
Traffic incidents	15,321	610
Nontraffic incidents	8,064	2,174
TOTAL	290,669	22,816

Table 2.—Number of vehicles involved in reported and unreported traffic accidents in Illinois during 1958, and the total direct cost of such accidents

Vehicle type	Number of vehicles involved in accidents	Percent of total	Total direct cost	Percent of total	Cost per involvement	Involvements per 10 million vehicle-miles [1]	Cost per vehicle-mile [2]
PASSENGER CARS							
Reported involvements	317,100	24.1	$149,198,000	57.7	$471	119	0.56
Unreported involvements	1,000,600	75.9	109,572,000	42.3	110	374	.41
TOTAL	1,317,700	100.0	258,770,000	100.0	196	493	.97
TRUCKS							
Single-unit trucks:							
Reported involvements	20,600	18.8	$5,818,000	43.3	$282	50	0.14
Unreported involvements	89,100	81.2	7,607,000	56.7	85	216	.19
Subtotal	109,700	100.0	13,425,000	100.0	122	266	.33
Truck combinations:							
Reported involvements	4,500	24.5	2,367,000	50.8	526	54	0.28
Unreported involvements	13,900	75.5	2,289,000	49.2	165	167	.28
Subtotal	18,400	100.0	4,656,000	100.0	253	221	.56
All types of trucks:							
Reported involvements	25,100	19.6	8,185,000	45.3	326	51	0.16
Unreported involvements	103,000	80.4	9,896,000	54.7	96	207	.20
TOTAL	128,100	100.0	18,081,000	100.0	141	258	.36

[1] Travel of Illinois registered vehicles: Passenger cars, 26,748,000,000 vehicle-miles; single-unit trucks, 4,124,000,000 vehicle-miles; and truck combinations, 832,000,000 vehicle-miles.
[2] Fraction of one cent.

erent in sampling single vehicle accidents and ultiple vehicle accidents and in tracing the wnership of vehicles involved in multivehicular accidents.

Scope of Study

As the primary purpose for undertaking udies of this type is to develop accident cost ata, a discussion of cost concepts is necessary. he theory upon which such studies are based, s developed by a committee of the Highway esearch Board in 1949, may be stated briefly s those costs represented by the money value f damages and losses to persons and property. Ioney spent by persons involved in accidents ay or may not be the same as the money alue of damages or losses. Damage to roperty may not be repaired and losses may ot be compensated for, but such costs are ncluded in the money value concept as they ill be realized in the form of depreciated alue or decreased earnings. Payment for amages and losses is not always made by the ehicle owner or person injured; the driver or wner of another vehicle may pay the costs; nsurance companies may reimburse in full or n part for damages; hospitals, doctors, and thers may furnish services and not be compensated fully; and courts may award damages n excess of or less than actual costs. No ttempt has been made here to trace the ransfer of money or to determine actual mounts of money spent, except to the extent hat such expenditures measure the money value of damages or losses to persons and roperty.

Direct costs are composed of the money value of: Damage to property, ambulance se, hospital and treatment services, doctor nd dentist services, loss of use of vehicle, value of work time lost, legal and court fees, damage awards and settlements, and other miscellaneous items. The valuation of these direct costs was made on the basis of information supplied by persons whose vehicles were involved in accidents, by persons who were injured in accidents, relatives of injured persons, doctors and dentists, insurance agents and brokers, attorneys, police, and others. A detailed explanation of the different cost elements considered in the study is given in reference *1*.

Such items as loss of future earnings of persons killed or permanently injured in accidents were excluded from the direct cost phase of the study, except to the extent that damage awards or settlements made either in or out of court might have compensated for such losses. Expenditures also excluded from the direct cost phase of the study were those made by public and private agencies in the interest of accident prevention or to mitigate the economic burden of accidents and the overhead cost of automobile and certain other types of insurance.

The summary in table 1 provides an overall perspective of total direct costs of accidents and incidents that occurred in Illinois during 1958, as determined in this study. Upon adding the cost out-of-State accidents and incidents of Illinois vehicles to the above data, total direct costs would be as follows: Passenger cars, $309.5 million; and trucks, $29.3 million. The costs thus determined in the study amounted to one-third of a billion dollars, or an average of $928,000 per day.

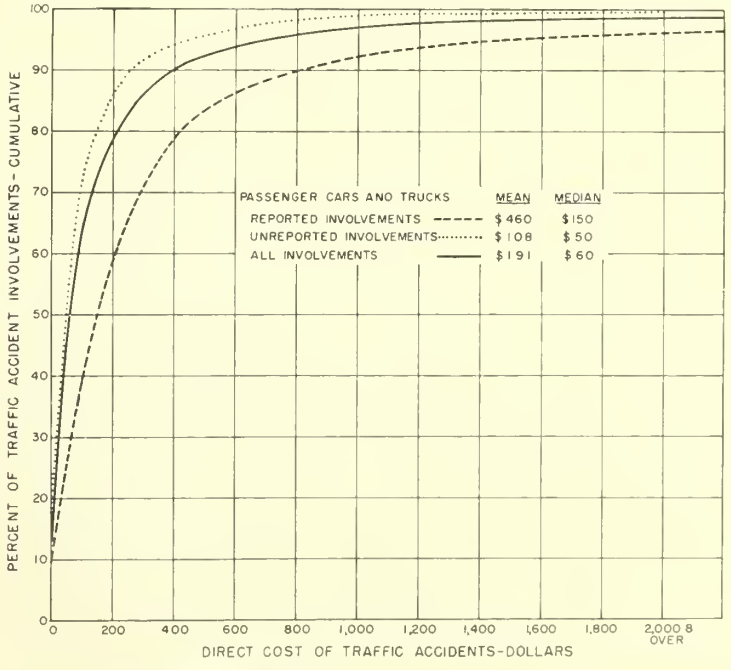


Figure 1.—Cumulative percentage distribution of reported and unreported traffic accident involvements, plotted in $100 direct-cost intervals.

Table 3.—Number of Illinois registered vehicles in use during 1958, and average annual in-State travel per vehicle

Vehicle type	Vehicles in use	Average annual travel
Passenger cars	2,876,000	9,300
Single-unit trucks		
Panels and pickups	194,400	11,950
Other 2-axle trucks	153,800	10,900
3-axle trucks	6,600	19,140
All single unit trucks	354,800	11,620
Truck combinations		
3-axle tractor semitrailers	12,000	24,850
Other truck combinations	11,300	47,340
All combinations	23,300	35,740

In order to avoid possible misconceptions, the fact is emphasized that the data do not include the cost of all accidents occurring on Illinois highways. Only direct costs to persons and property associated with accidents or incidents involving privately owned Illinois registered passenger cars and trucks have been included. Specifically, the data are representative of the costs incurred by owners and occupants of Illinois passenger cars and trucks, pedestrians, and other nonmotorists involved in such accidents. Direct costs excluded from the study were those to persons and property associated with accidents that involved: (1) Out-of-State registered motor vehicles of all types, (2) publicly owned motor vehicles of all types, and (3) Illinois registered buses, motorcycles, motorized bicycles and scooters, and any special purpose vehicles. Costs incurred by owners and occupants of these three categories of vehicles have been excluded even though such vehicles may have been involved in an accident with a privately owned Illinois passenger car or truck.

Although the study encompassed total accident and incident experience of Illinois passenger car and truck owners, regardless of whether the events occurred on or off the highway or in or out of State, subsequent discussion in this article is restricted to traffic accidents occurring on Illinois highways and streets.

Table 4.—Distribution of Illinois registered vehicles involved in traffic accidents and the corresponding direct costs, by severity of accident

Severity of accident	Distribution of accident involvements and costs	
	Percent of vehicles involved	Percent of cost
Passenger cars:		
Fatal injury	0.1	3.1
Nonfatal injury	12.5	52.2
Property damage only	87.4	44.7
TOTAL	100.0	100.0
Trucks:		
Fatal injury	0.2	6.7
Nonfatal injury	7.6	37.2
Property damage only	92.2	56.1
TOTAL	100.0	100.0

Table 5.—Number of traffic accident involvements in Illinois involving vehicles of Illino registry, 1958, classified by severity of accident and cost elements incurred

Cost element	Passenger car accidents				Truck accidents			
	Fatal injury	Nonfatal injury	Property damage only	Total	Fatal injury	Nonfatal injury	Property damage only	Total
Number of involvements having:								
Damage to vehicle	1,391	142,824	990,672	1,134,887	189	6,001	66,639	72,829
Damage to property in vehicle	75	2,708	12,929	15,712	44	384	2,158	2,586
Damage to objects struck by vehicle	85	2,067	17,252	19,404	18	195	3,295	3,508
Miscellaneous property damage	28	3,450	5,263	8,741	27	137	1,488	1,652
Involvements having one or more property damage cost elements	1,391	143,259	1,000,539	1,145,189	193	6,087	68,539	74,819
Ambulance costs	625	7,224	-------	7,849	55	761	------	816
Doctor and dentist fees	903	84,104	-------	85,007	105	2,827	------	2,932
Hospital and treatment costs	940	64,188	-------	65,128	93	2,089	------	2,182
Miscellaneous injury costs	334	6,885	-------	7,219	27	261	------	288
Involvements having one or more injury cost elements	1,067	94,703	-------	95,770	119	2,967	------	3,086
Loss of use of vehicle costs	43	6,473	23,037	29,553	55	780	5,796	6,631
Value of time lost from work	653	77,368	22,817	100,838	94	2,428	3,612	6,134
Legal and court costs	734	37,296	10,108	48,138	90	1,342	644	2,076
Damage awards in excess of known costs	705	48,810	9,227	58,742	109	1,130	96	1,335
SUMMARY:								
Involvements having one or more direct cost elements	1,532	155,057	1,003,041	1,159,630	232	6,718	69,882	76,832
Involvements incurring no costs	28	9,534	148,466	158,028	5	2,955	48,293	51,253
Total involvements	1,560	164,591	1,151,507	1,317,658	237	9,673	118,175	128,085

Reported and Unreported Accident Involvements

Data included in table 2 show the relationship of reported and unreported accident involvements and the corresponding costs. An unreported involvement refers to an event for which no record of an owner's report could be found in the accident report files maintained by the Illinois Division of Highways. Several factors could account for this, but the principal one would be that property damage costs were less than the legal reporting minimum. If the accident were of the reportable category and no record could be found, one of the following conditions might apply: The owner may have reported the accident to local authorities but not to the State; the owner may have failed to report the happening to any governmental authority; or through error the accident report may have been overlooked in the search of the State's accident files. Every effort was made to prevent the latter possibility through a careful review of all reportable accidents.

Approximately 1.3 million Illinois passenger cars of private ownership were involved in traffic accidents on Illinois roads and streets during 1958. Direct costs of these accidents amounted to $258.8 million or an average of $196 per passenger car involved. Totals include all degrees of severity—fatal, nonfatal, and property-damage-only accidents. Three-fourths of these events were not officially reported to the Illinois Division of Highways, and in the aggregate they accounted for more than two-fifths of the total cost. The mean value for unreported passenger car involvements was $110 and the median value was $50.

Approximately 128,000 trucks were i volved in accidents costing $18.1 million, an average of $141 for each event. Unr ported involvements accounted for four-fift of the number and more than one-half of t total cost. The mean and median values f unreported truck involvements were $96 an $20, respectively.

It should not be construed that all unr ported involvements in which costs exceede $100 were in violation of the reporting la The cost values include elements that do n enter into the legal reporting requirement damage to property. For example, suc elements as time lost from work or loss use of vehicle are included when applicab in the cost values shown in table 2.

The cost distribution of reported and u reported involvements is illustrated in figu 1. It is clearly evident that a very hig proportion of unreported involvements we relatively minor events. Ninety-two perce of these unreported events cost less than $3 each. The same percentage for official reported involvements indicated costs of le than $1,000.

Accident Exposure

Accident involvement rates for passeng cars calculated on the basis of 10 millio vehicle-miles of travel, as shown in table were nearly twice those for trucks, and t cost of accidents per vehicle-mile of trav approached 1 cent for passenger cars, 2 times the rate for trucks. When trucks we considered on the basis of single units a combinations, the data showed a lower i volvement rate for combinations but a high cost per vehicle-mile. This relationship cou

gically be expected as most operators of uck combinations would be more experienced nd skillful drivers. Vehicle and vehicle-use haracteristics should also be considered in ich a comparison. On the other hand, hen the heavy units were involved in acci-ents, they tended to be more severe and ostly, particularly when cargo damage was nvolved. Among the single-unit trucks, anels and pickups accounted for 55 percent f the vehicles in use, 56 percent of the travel, nd 53 percent of the single-unit vehicles nvolved in accidents. These two truck types re often used for personal transportation, nd in many respects their operation is similar o that of passenger cars.

Privately owned Illinois vehicles registered nd in use during 1958 and their average nnual in-State travel per vehicle (*2*) are hown in table 3. In relating vehicles in use o the number of vehicles involved in ac-idents, it was found that the probability of a assenger car being involved in a traffic acci-ent was once in 26 months; for single-unit rucks, once in 39 months; and for truck ombinations, once in 15 months. Exposure o accidents, based on average annual travel, vas three times greater for truck combinations han for single-unit trucks, and nearly four imes greater than for passenger cars.

Table 6.—Direct cost of traffic accidents in Illinois involving vehicles of Illinois registry, 1958, classified by severity of accident and cost elements incurred

Cost element	Direct cost of passenger car accidents				Direct cost of truck accidents			
	Fatal injury	Nonfatal injury	Property damage only	Total	Fatal injury	Nonfatal injury	Property damage only	Total
Property damage:								
Damage to vehicle	$1,196,385	$41,368,456	$109,795,996	$152,360,837	$270,836	$2,191,845	$7,642,290	$10,104,971
Damage to property in vehicle	8,225	160,670	645,458	814,353	38,222	80,001	171,903	290,126
Damage to objects struck by vehicle	23,218	406,368	1,688,634	2,118,220	2,368	164,805	704,232	871,405
Miscellaneous property damage	846	69,548	142,302	212,696	1,761	6,095	17,267	25,123
Subtotal	1,228,674	42,005,042	112,272,390	155,506,106	313,187	2,442,746	8,535,692	11,291,625
Treatment of injuries:								
Ambulance costs	19,317	173,300	--------	192,617	1,495	17,234	------	18,729
Doctor and dentist fees	354,709	10,304,366	--------	10,659,075	25,325	615,569	------	640,894
Hospital and treatment costs	686,858	9,415,140	--------	10,101,998	32,178	339,578	------	371,756
Miscellaneous injury costs	29,845	318,974	--------	348,819	1,246	11,395	------	12,641
Subtotal	1,090,729	20,211,780	--------	21,302,509	60,244	983,776	------	1,044,020
Loss of use of vehicle costs	10,152	666,718	1,013,342	1,690,212	61,697	236,266	1,446,990	1,744,953
Value of time lost from work	636,239	17,274,842	846,022	18,757,103	63,436	1,688,287	129,008	1,880,731
Legal and court costs	1,557,909	23,301,020	1,091,790	25,950,719	146,509	542,818	28,056	717,383
Damage awards in excess of known costs	3,372,203	31,655,984	534,784	35,562,971	570,297	830,934	1,301	1,402,532
TOTAL COST	7,895,906	135,115,386	115,758,328	258,769,620	1,215,370	6,724,827	10,141,047	18,081,244

Direct Cost Elements

The cost elements that make up the total ost figures shown in table 2 are shown in onsiderable detail in tables 5–7. The relative umber and cost of each of the three severity lasses of accidents are shown in table 4. t is evident that fatal injury involvements ccounted for a small proportion of the num-ber and cost of accidents. Also, nonfatal njury accidents involving passenger cars epresented a considerably higher proportion f the total costs than similar events involving rucks. Injuries to passengers would largely ccount for this difference. Trucks normally have only one occupant, the driver.

As mentioned earlier, the cost data do not include values for the loss of future earnings f persons killed or permanently injured, ex-cept to the extent that awards or settlements may measure this loss. Awards or settle-ments are based primarily on the fault concept, and thus the victim or survivors may not have recourse to recover losses caused by death or injury. This situation would apply particularly to single vehicle accidents.

Passenger car and truck involvements that occasioned no costs (or less than $5.00) were very numerous as indicated in table 5. A comparison of such events is shown in table 8. The finding that approximately 2 percent of the fatal injury involvements were of the no cost category might appear unreasonably high at the outset. A typical case would be a passenger car or truck colliding with a pedestrian. Assume that the pedestrian was at fault, that the victim died instantly, that the vehicle was not damaged, that no time was lost from work by the vehicle owner or driver, and that a police vehicle was used to remove the victim from the scene. Under the conditions just outlined, no costs would be assessed for this accident within the scope of the direct cost phase of the study. Funeral costs are not considered as an element of cost in connection with a motor-vehicle accident. Such costs are inevitable; an accident merely fixes the time when they are incurred.

Another example of a no cost involvement applies to a multiple vehicle accident. In a two-car collision, one vehicle might be damaged and the bumper of the other vehicle absorbs the shock. Under the sampling procedure used in the study, either vehicle or both might be selected. A large proportion

Table 7.—Mean values for cost elements incurred in Illinois traffic accidents involving vehicles of Illinois registry, classified by severity of accident

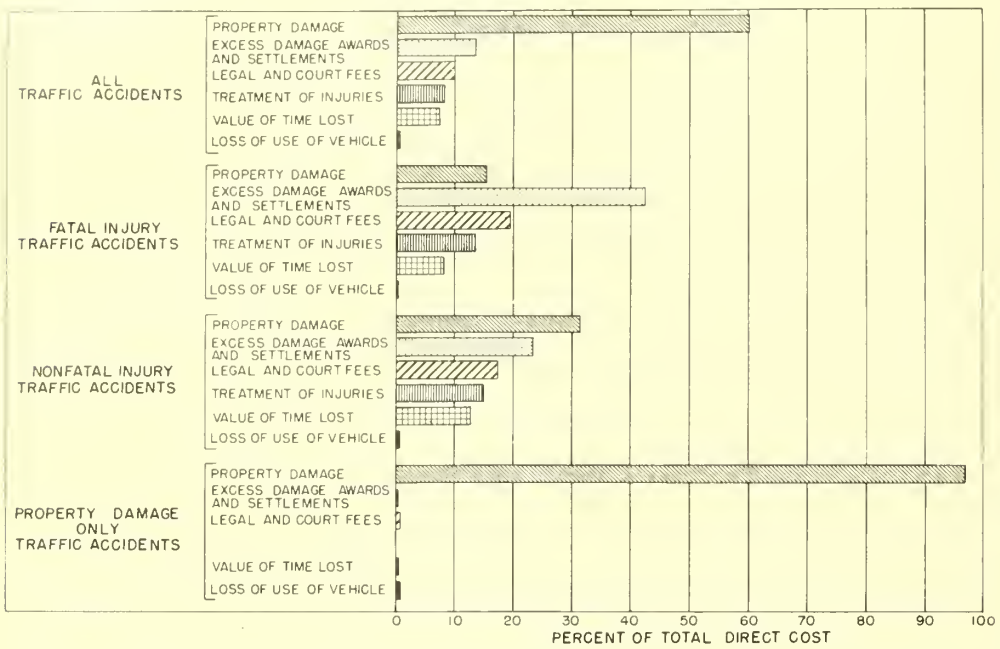
Cost element	Mean cost values for each element of cost incurred in—							
	Passenger car traffic accidents				Truck traffic accidents			
	Fatal injury	Nonfatal injury	Property damage only	All severity classes	Fatal injury	Nonfatal injury	Property damage only	All severity classes
Property damage:								
Damage to vehicle	$860	$290	$111	$134	$1,433	$365	$115	$139
Damage to property in vehicle	110	59	50	52	869	208	80	112
Damage to objects struck by vehicle	273	197	98	109	132	845	214	248
Miscellaneous property damage	30	20	27	24	65	44	12	15
Mean cost value for involvements in which one or more property damage cost elements were incurred	883	293	112	136	1,623	401	125	151
Treatment of injuries:								
Ambulance costs	31	24	------	25	27	23	------	23
Doctor and dentist fees	393	123	------	125	241	218	------	219
Hospital and treatment costs	731	147	------	155	346	163	------	170
Miscellaneous injury costs	89	46	------	48	46	44	------	44
Mean cost value for involvements in which one or more injury cost elements were incurred	1,022	213	------	222	506	332	------	338
Loss of use of vehicle costs	236	103	44	57	1,122	303	250	263
Value of time lost from work	974	223	37	186	675	695	36	307
Legal and court costs	2,122	625	108	539	1,628	404	44	346
Damages awards in excess of known costs	4,783	649	58	605	5,232	735	14	1,051
Mean cost value for involvement in which one or more cost elements were incurred	5,154	871	115	223	5,239	1,001	145	235

Table 8.—Traffic accident involvements, in which no costs were incurred, related to severity of involvements

Severity of accident	No cost involvements as a percent of severity classes	
	Passenger cars	Trucks
	Percent	*Percent*
Fatal injury	1.8	2.1
Nonfatal injury	5.7	30.5
Property damage only	12.9	40.9
All severity classes	12.0	40.0

Table 9.—Samples sizes compared with expanded number of traffic accident involvements

Severity of accident	Number of involvements	Number of sample cases
Passenger cars:		
Fatal injury	1,560	332
Nonfatal injury	164,591	1,761
Property damage only	1,151,507	1,290
TOTAL	1,317,658	3,383
Trucks:		
Fatal injury	237	200
Nonfatal injury	9,673	1,270
Property damage only	118,175	1,556
TOTAL	128,085	3,026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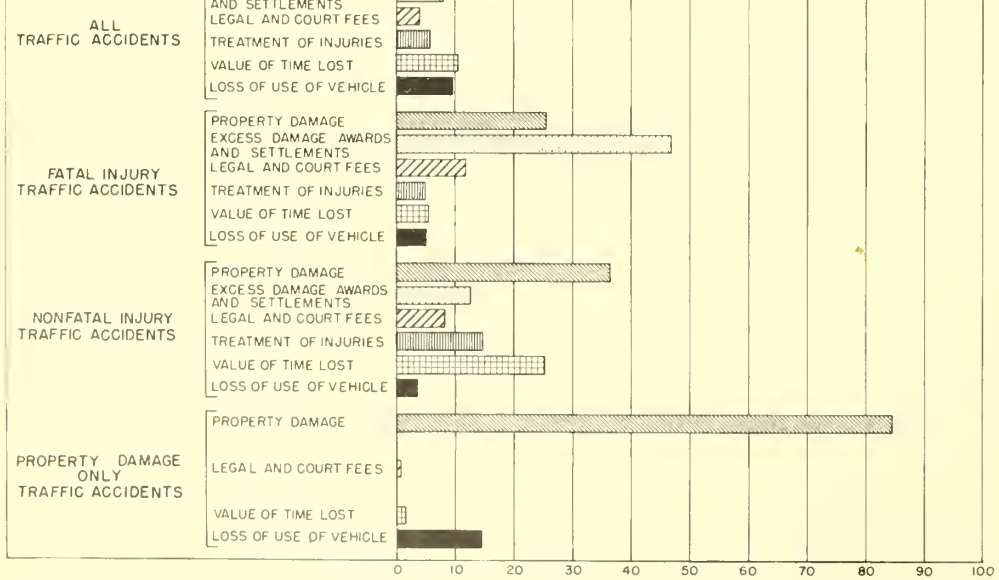
NONFATAL INJURY TRAFFIC ACCIDENTS
PROPERTY DAMAGE ONLY TRAFFIC ACCIDENTS
PROPERTY DAMAGE
EXCESS DAMAGE AWARDS AND SETTLEMENTS
LEGAL AND COURT FEES
TREATMENT OF INJURIES
VALUE OF TIME LOST
LOSS OF USE OF VEHICLE
0 10 20 30 40 50 60 70 80 90 100
PERCENT OF TOTAL DIRECT COST

Figure 2.—Percentage distribution of the direct costs of passenger car traffic accidents, by cost element.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PROPERTY DAMAGE
EXCESS DAMAGE AWARDS AND SETTLEMENTS
LEGAL AND COURT FEES
TREATMENT OF INJURIES
VALUE OF TIME LOST
LOSS OF USE OF VEHICLE
0 10 20 30 40 50 60 70 80 90 100
PERCENT OF TOTAL DIRECT COST

Figure 3.—Percentage distribution of the direct costs of truck traffic accidents, by cost element.

of the no cost involvements were of the unre ported accident category, as illustrated i figure 1. Trucks, in particular, were involve in a number of nonfatal injury and propert damage only accidents in which no costs wer incurred by the owner or occupants of th vehicle selected. This situation is explaine partially by the fact that most truck accident involved collisions with passenger cars. Con ditions acting in favor of trucks from the cos standpoint were the lower occupancy rat (persons per vehicle) and vehicle capabilit to withstand impact. The severity classifica tion is determined by the accident and no by what takes place in one of the vehicle involved.

In a study based upon sampling techniques it is obvious that the greater the detail pro vided in tabular form the greater the chanc of exceeding the built-in limitations of sampl size. As an indication of the strength of th data reported in tables 5–7, a comparison o sample sizes and expanded totals is provide in table 9.

The total cost figure of $258.8 million fo passenger car accidents, reported in table 6 is based upon 3,383 completed sample cases and the amount of $18.1 million for trucks i based upon 3,026 cases. The ratios of sampl cases to the expanded number of involvement do not reflect the sampling rates as originall selected. As mentioned earlier, two samplin sources were used—official accident report and registration lists—and different samplin rates applied. A full description of samplin procedures is given in reference *1*.

Cost data shown in table 6 are furthe illustrated in figures 2 and 3. The top set o bars in figure 2, arranged in order of magni tude, shows the distribution of the accident dollar. Property damage accounted for 60 percent (60 cents of the accident dollar) o he total cost of all passenger car traffi accidents, and 62 percent of all truck traffi accidents (fig. 3). Trea ment of injuries legal and court fees, and excess damage awards and settlements accounted for a large proportion of the total cost of passenger ca accidents than for trucks. On the other hand costs related to time loss and loss of use o vehicle represented a larger proportion of th total cost for trucks than for passenger cars The cost element "loss of use of vehicle" i not too significant in the case of passenger ca owners because in most cases the use of th vehicle is not essential in earning a livelihood The latter criterion is used in determinin such costs.

For truck owners, and particularly fleet operators, no loss of use of vehicle costs have been included when standby equipment was available to replace the damaged vehicle Only a part of the cost of maintaining standby equipment could properly be charged t motor-vehicle accidents as standby vehicles are brought into service for purposes othe than accidents; such as, peak operations maintenance of equipment, etc. The pro rata share of the overhead cost of maintaining standby equipment to be charged to accidents would be included in the indirect cost phase of accident cost studies.

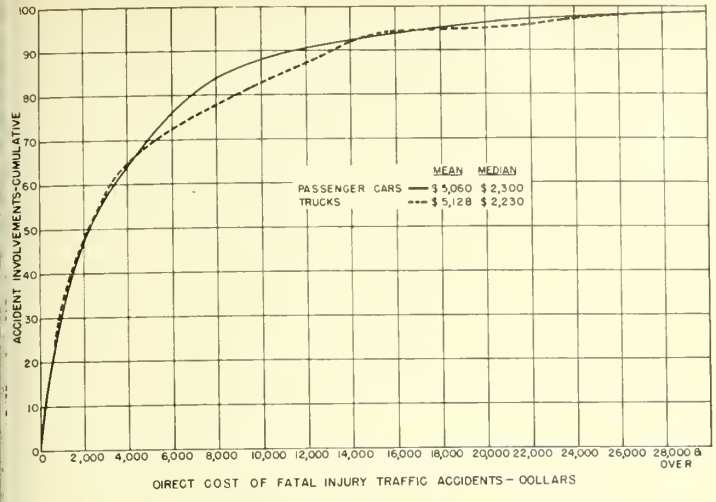

ACCIDENT INVOLVEMENTS-CUMULATIVE
MEAN MEDIAN
PASSENGER CARS — $5,060 $2,300
TRUCKS --- $5,128 $2,230
DIRECT COST OF FATAL INJURY TRAFFIC ACCIDENTS – DOLLARS

Figure 4.—Cumulative percentage distribution of passenger car and truck fatal injury traffic accident involvements, plotted in $1,000 direct-cost intervals.

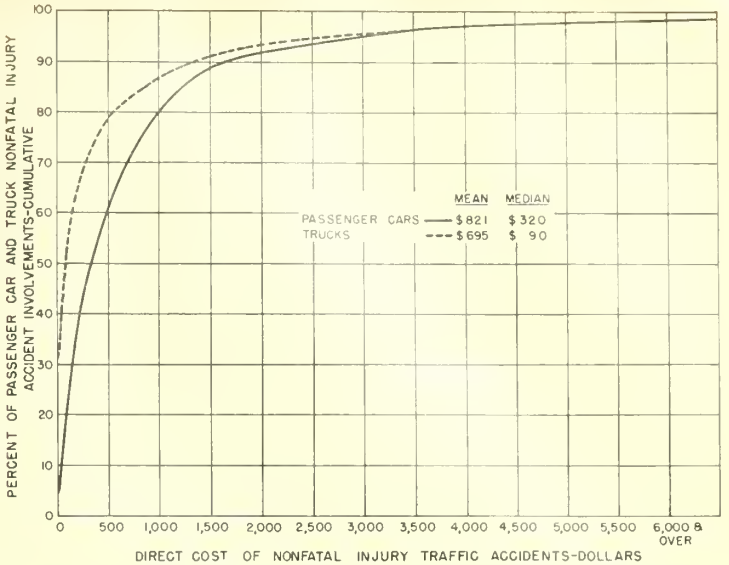

PERCENT OF PASSENGER CAR AND TRUCK NONFATAL INJURY ACCIDENT INVOLVEMENTS-CUMULATIVE
MEAN MEDIAN
PASSENGER CARS — $821 $320
TRUCKS --- $695 $ 90
DIRECT COST OF NONFATAL INJURY TRAFFIC ACCIDENTS-DOLLARS

Figure 5.—Cumulative percentage distribution of passenger car and truck nonfatal injury traffic accident involvements, plotted in $100 direct-cost intervals.

Damage awards and settlements in excess of known costs represented the greatest part of the accident dollar for both passenger car and truck fatal injury accidents. In determining excess awards and settlements, compensation received by each injured person or survivor and by each vehicle owner was considered on an individual basis. Payments received by the injured person or vehicle owner from his own insurance company were not considered as awards or settlements, as such payments would simply represent a return of capital. Damage awards and settlements include payments made by the other party, presumably the one found liable. Lump-sum payments under workmen's compensation were included also.

In the case of an injured person, known costs of ambulance use, hospitalization, doctor and dentist fees, time loss, legal fees, etc., were deducted from the award or settlement, and any surplus represented reimbursement for costs that could not be classified. A vehicle owner may also receive a settlement for damage to his vehicle, other property, time loss, loss of use of vehicle, etc., and the settlement may exceed the known costs. The surplus again was treated as an unclassified cost.

In the study procedure, awards and out-of-court settlements were recorded in total, regardless of whether the amounts were less than, equal to, or greater than the actual money value of damages and losses. Obviously, the total amount of an award or settlement could not be added to the previously determined money value of damages and losses as this procedure would duplicate all or part of the costs. For this reason, the amount of damage awards and settlements was ascertained, but only the part that was in excess of the value of damages and losses was included in the cost of accidents. Such excess awards or settlements could represent compensation for pain and suffering, loss of future earnings of persons killed or permanently injured, future medical expenses, and other indeterminable costs.

Mean values for each element of cost incurred in passenger car and truck accidents, as reported in table 7, were heavily influenced by high cost accidents. Median values for each cost element would be substantially lower than the values reported. The positive skewness of the cost curves for each of the severity classes of accidents is illustrated later.

The final entry in table 7 indicates the average costs of accident involvements in which one or more cost elements were incurred. Truck involvements for each severity class averaged higher costs than was the case for passenger cars. Costs sustained in traffic accidents of all severity classes averaged $223 for passenger car involvements and $235 for truck involvements. After including involvements in which no costs were incurred, as reported in table 5, the averages dropped to $196 and $141, respectively.

Skewness of Cost Distribution

The difficulties of sampling the universe of traffic accident involvements for the purpose of determining cost data are apparent after viewing the cumulative percentage curves in figures 4–7. Findings of the study show a range of costs per vehicle involvement from zero (or less than $5) to $136,800. Figure 4

Table 10.—Number of vehicles involved in traffic accidents and the direct costs of such accidents, classified by vehicle type and accident location

Vehicle type	Rural areas	Municipality populations				
		Under 5,000	5,000—24,999	25,000—125,000	1,000,000 and over	All municipalities
NUMBER OF VEHICLES INVOLVED IN TRAFFIC ACCIDENTS						
Passenger cars	190,975	77,463	234,189	302,828	512,203	1,126,683
Trucks:						
Single-unit:						
Panels and pickups	9,376	7,539	6,663	11,412	22,095	47,709
Other single-unit trucks	13,172	2,345	5,806	9,985	19,789	37,925
All single-unit trucks	22,548	9,884	12,469	21,397	41,884	85,634
Truck combinations	3,781	1,049	1,797	3,220	8,506	14,572
Unknown truck type	487	--------	102	493	468	1,063
All single-unit trucks and truck combinations	26,816	10,933	14,368	25,110	50,858	101,269
DIRECT COST OF TRAFFIC ACCIDENTS						
Passenger cars	$60,981,882	$11,324,294	$29,745,538	$45,289,744	$111,428,162	$197,787,738
Trucks:						
Single-unit:						
Panels and pickups	4,046,099	552,199	507,201	1,025,711	1,494,471	3,579,582
Other single-unit trucks	2,991,158	291,468	305,192	832,470	1,309,983	2,739,113
All single-unit trucks	7,037,257	843,667	812,393	1,858,181	2,804,454	6,318,695
Truck combinations	2,059,289	522,112	516,291	347,532	1,211,188	2,597,123
Unknown truck type	9,963	--------	3,057	40,528	15,332	58,917
All single-unit trucks and truck combinations	9,106,509	1,365,779	1,331,741	2,246,241	4,030,974	8,974,735

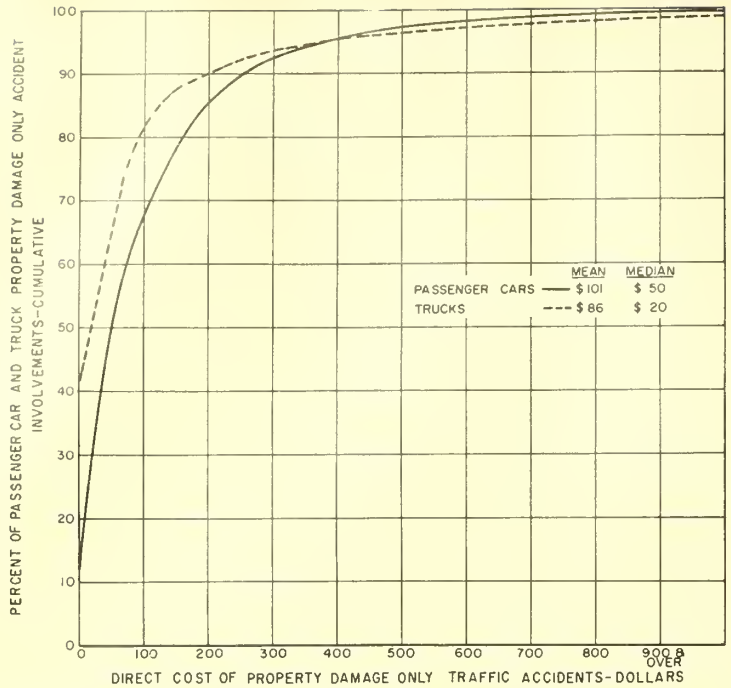

PERCENT OF PASSENGER CAR AND TRUCK PROPERTY DAMAGE ONLY ACCIDENT INVOLVEMENTS-CUMULATIVE
MEAN MEDIAN
PASSENGER CARS $101 $ 50
TRUCKS $86 $ 20
DIRECT COST OF PROPERTY DAMAGE ONLY TRAFFIC ACCIDENTS-DOLLARS

Figure 6.—Cumulative percentage distribution of passenger car and truck property damage only traffic accident involvements, plotted in $50 direct-cost intervals.

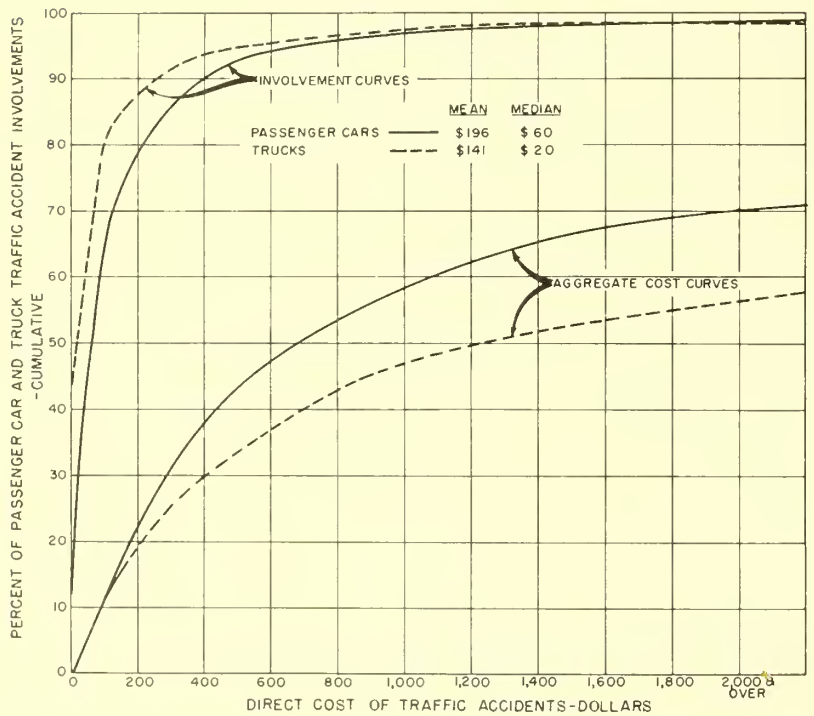

PERCENT OF PASSENGER CAR AND TRUCK TRAFFIC ACCIDENT INVOLVEMENTS -CUMULATIVE
INVOLVEMENT CURVES
MEAN MEDIAN
PASSENGER CARS $196 $ 60
TRUCKS $141 $ 20
AGGREGATE COST CURVES
DIRECT COST OF TRAFFIC ACCIDENTS-DOLLARS

Figure 7.—Cumulative percentage distribution of passenger car and truck traffic accident involvements and aggregate costs, plotted in $100 direct-cost intervals.

illustrates the case in point. Ninety percent of the fatal injury passenger car involvements fell within the cost range of $11,600 or less; a similar percentage for trucks indicated a range of $13,200 or less. The remaining 10 percent of the fatal injury passenger car involvements accounted for 48 percent of the total direct costs of fatal injury accidents. For trucks, the same proportionate group accounted for 45 percent of the total direct costs of fatal injury accidents. The extreme plotting interval in figure 4 of $28,000 and over was representative of only 1½ percent of the total fatal injury involvements for both passenger cars and trucks, and yet this remote class accounted for nearly 19 percent of the costs of fatal injury passenger car accident and nearly 12 percent of the total in th case of trucks.

The cumulative percentage curves for non fatal injury accident involvements are illus trated in figure 5. Again the extreme plottin interval of $6,000 and over was representativ of 1½ percent of both passenger car and truc nonfatal injury involvements. This grou however, accounted for 26 percent of the tota cost of nonfatal injury passenger car involve ments and 22 percent of the total in the cas of trucks.

As would be expected, the range in costs o property damage only involvements was les extreme than was found for fatal and nonfata injury involvements. There are exceptiona cases though. A heavily damaged passenge car usually causes injury to the driver or passenger. Trucks, on the other hand, ma run off the highway, overturn, and caus excessive damage to vehicle and load, but th driver may escape unscathed. The plottin interval of $900 and over, shown in figure 6 accounted for 0.5 percent of the passenge car involvements and slightly over 1 percen for trucks. Costs represented by these smal groups accounted for 5 percent of the total fo passenger cars and 25 percent of the tota for trucks.

As a further indication of the extreme cos values found in the study, fatal injury involve ments ranged from zero to $136,800 for pas senger cars and from zero to $46,200 fo trucks. Nonfatal injury involvements range from zero to $73,300 for passenger cars an to $53,700 for trucks. Property damage onl involvements reached a maximum of $1,40 for passenger cars and $30,100 for trucks.

High cost accident cases found in th Illinois study pointed to the need for furthe refinement in sample design. The extent o such refinement in sample design depend largely upon the data available on tabulatin cards in a given State's files of officially re ported accidents. Of necessity, the samplin procedures in the past have been adapted t existing records.

Composite involvement and aggregate cos curves for all severity classes of involvement are shown in figure 7. The average or mea value for passenger car involvements was $19 and for trucks was $141. The midvalues o medians were considerably less—$60 and $20 The cost interval of $2,000 and over, plotte at the extreme right of figure 7, represent only 1 percent of the total of 1.3 millio passenger car involvements and 30 percen of the total direct costs of $258.8 million An identical comparison for trucks indicate that 1½ percent of the 128,100 involvement fell within the cost interval of $2,000 and over and this group accounted for 44 percent o the $18.1 million total.

By selecting the cost interval of $10,00 and over, generally the lower limit for bodily injury and liability insurance, 0.1 percent o 1,339 passenger car involvements out of the total of 1,317,700 and 0.07 percent or 90 o the truck involvements out of a total o 128,100 fell into this cost interval. These relatively few involvements, however, account ed for 10 and 11 percent, respectively, of the

otal direct cost of passenger car and truck ccidents.

On the basis of the above Statewide comparison, and assuming that 1958 experience f Illinois owners was typical, the chance of passenger car owner being involved in an ccident in which the costs associated with is vehicle would amount to $10,000 or more ould be about 1 in 1,000; for truck owners, he probability of such an event would be bout 1 in 1,400. As indicated previously, 2,876,000 Illinois passenger cars were driven he equivalent of 26.7 billion vehicle-miles in 958. By referring again to the 1,339 passenger car involvements in which costs qualled or exceeded $10,000, it is evident hat the frequency of such an occurrence ould be 5.0 involvements per 100 million ehicle-miles, or 1.0 involvement per 20 million ehicle-miles. On this basis, one of approximately forty passenger car owners in a lifeime of vehicle ownership would be expected o experience an accident in which the costs ssociated with his vehicle would equal or xceed $10,000.

Data included in figures 8–10 show the cost istribution of fatal, nonfatal, and property amage only involvements on the basis of the umber of involvements rather than percent f involvements as illustrated in figures 4–7. he bars in figures 8–10 are representative of he combined number of passenger car and uck involvements. Figure 11 represents a omposite distribution for all severity classes f involvements. Many of the characteristics f the cost distribution for each of the severity lasses have already been mentioned and need o further emphasis. The bar charts, however, illustrate more forcefully the positive kewness of accident cost curves and emphaize the inherent problems in sampling the niverse of accident involvements for the urpose of determining costs. Obviously, he high cost involvements are subject to onsiderable sampling variability.

Frequencies and Costs of Accident Involvements Related to Accident Location

The usual approach in determining accident xposure is to relate the number of accidents o vehicle-miles of travel. Fortunately the notor-vehicle-use study, conducted by the llinois Division of Highways during 1958, omplements the motor-vehicle accident cost tudy. The availability of this information is n invaluable aid in relating accidents to ighway- and vehicle-use characteristics.

Data included in tables 10 and 11 provide he basis for determining the frequencies and osts of accident involvements occurring in ural areas and municipalities. The term unicipality is used to denote incorporated laces regardless of population size. Unincorporated places are included in the rural lassification.

Numbers of vehicles involved in traffic ccidents and the corresponding costs are not oo meaningful unless such events can be elated to exposure. Involvement and cost ates per 10 million vehicle-miles of travel are eported in table 12 for passenger cars and

Table 11.—Vehicle-miles of travel in Illinois by vehicles of different types, classified by the location of travel [1]

Vehicle type	Vehicle-miles of travel (1,000)					
	Rural areas	Municipality populations				
		Under 5,000	5,000–24,999	25,000–125,000	1,000,000 and over	Total
Passenger cars	9,986,084	1,984,221	3,012,843	4,064,738	7,700,420	16,762,222
Trucks:						
Single-unit:						
Panels and pickups	1,239,747	176,595	236,846	246,216	422,536	1,082,193
Other single-unit trucks	1,072,841	125,410	144,528	149,807	309,226	728,971
All single-unit trucks	2,312,588	302,005	381,374	396,023	731,762	1,811,164
Truck combinations	521,188	63,672	60,389	42,480	144,929	311,470
All single-unit trucks and truck combinations	2,833,776	365,677	441,763	438,503	876,691	2,122,634

[1] Data represent travel of Illinois registered vehicles in use. Source: *Motor Vehicle Use Study*, State of Illinois, Department of Public Works and Buildings, Division of Highways, October 1961.

Table 12.—Number of vehicles involved in traffic accidents and the direct costs of such accidents, per 10 million vehicle-miles of travel, classified by vehicle type and accident location

Vehicle type	Rural areas	Municipality populations				
		Under 5,000	5,000–24,999	25,000–125,000	1,000,000 and over	All municipalities
NUMBER OF VEHICLES INVOLVED IN TRAFFIC ACCIDENTS PER 10 MILLION VEHICLE-MILES						
Passenger cars	191	390	777	745	665	672
Trucks:						
Single-unit:						
Panels and pickups	76	427	281	463	523	441
Other single-unit trucks	123	187	402	667	640	520
All single-unit trucks	98	327	327	540	572	473
Truck combinations	73	165	298	758	587	468
All single-unit trucks and truck combinations	95	299	325	573	580	477
DIRECT COST OF TRAFFIC ACCIDENTS PER 10 MILLION VEHICLE-MILES						
Passenger cars	$61,067	$57,072	$98,729	$111,421	$144,704	$117,996
Trucks:						
Single-unit:						
Panels and pickups	32,636	31,269	21,415	41,659	35,369	33,077
Other single-unit trucks	27,881	23,241	21,116	55,569	42,363	37,575
All single-unit trucks	30,430	27,936	21,302	46,921	38,325	34,887
Truck combinations	39,511	82,000	85,494	81,811	83,571	83,383
All single-unit trucks and truck combinations	32,136	37,349	30,146	51,225	45,979	42,281

Table 13.—Number of municipalities and population, by city size groups in Illinois

Population group	Number of cities	Population, 1958
Under 5,000	1,026	1,135,700
5,000–24,999	138	1,399,500
25,000–125,000	33	1,750,100
1,000,000 and over	1	3,614,100
Subtotal	1,198	7,899,400
Rural areas		1,762,700
TOTAL	1,198	9,662,100

Table 14.—Number and cost of traffic accidents in Cook and Du Page Counties, Ill., during 1958

Street system	Rates per 10 million vehicle-miles	
	Number of accidents	Direct costs
Expressways	51	$30,800
Arterials	243	107,200
Local streets	1,021	309,400
All systems	347	132,400

major classes of trucks. Passenger car involvement rates ranged from 191 per 10 million vehicle-miles of travel in rural areas to 672 in municipalities of all population sizes, or a ratio of 1 accident involvement in ruaal areas for every 3.5 involvements in municipalities. For single-unit trucks, the ratio was 1 to 4.8; and for truck combinations, 1 to 6.4.

Direct costs of accident involvements per 10 million vehicle-miles of travel are shown in the lower half of table 12. On the basis of relative exposure, the cost of passenger car involvements ranged from $61,100 per 10 million vehicle-miles in rural areas to $118,000 in municipalities. Similar comparisons for single-unit trucks indicated a range of $30,400

Table 15.—Number of vehicles involved in traffic accidents and the direct costs of such accidents, classified by major vehicle type and highway system

Highway systems	Illinois registered passenger cars			Illinois registered single-unit trucks [1]			Illinois registered truck combinations			Illinois registered trucks, all types		
	Rural	Municipal	Total	Rural	Municipal	Total	Rural	Municipal	Total	Rural	Municipal	Total
NUMBER OF VEHICLES INVOLVED IN TRAFFIC ACCIDENTS												
Federal-aid primary and State highways	88,809	221,656	310,465	10,855	17,485	28,340	3,082	5,257	8,339	13,937	22,742	36,679
Federal-aid secondary:												
State highways	19,876	5,928	25,804	788	667	1,455	36	33	69	824	700	1,524
Local roads	14,166	4,135	18,301	1,714	24	1,738	38		38	1,752	24	1,776
Subtotal	34,042	10,063	44,105	2,502	691	3,193	74	33	107	2,576	724	3,300
Non-Federal-aid:												
State highways	9,330	111,402	120,732	1,534	12,165	13,699	252	2,025	2,277	1,786	14,190	15,976
Local roads	58,794	783,562	842,356	8,144	56,356	64,500	373	7,257	7,630	8,517	63,613	72,130
Subtotal	68,124	894,964	963,088	9,678	68,521	78,199	625	9,282	9,907	10,303	77,803	88,106
All roads and streets:												
State highways	118,015	338,986	457,001	13,177	30,317	43,494	3,370	7,315	10,685	16,547	37,632	54,179
Local roads	72,960	787,697	860,657	9,858	56,380	66,238	411	7,257	7,668	10,269	63,637	73,906
TOTAL	190,975	1,126,683	1,317,658	23,035	86,697	109,732	3,781	14,572	18,353	26,816	101,269	128,085
DIRECT COST OF TRAFFIC ACCIDENTS												
Federal-aid primary and State highways	$34,089,866	$45,582,939	$79,672,805	$4,543,900	$1,305,646	$5,849,546	$1,510,563	$1,743,175	$3,253,738	$6,054,463	$3,048,821	$9,103,284
Federal-aid secondary:												
State highways	3,292,274	1,270,202	4,562,476	144,672	95,649	240,321	4,641	788	5,429	149,313	96,437	245,750
Local roads	4,611,364	327,622	4,938,986	429,755	231	429,986	315,935		315,935	745,690	231	745,921
Subtotal	7,903,638	1,597,824	9,501,462	574,427	95,880	670,307	320,576	788	321,364	895,003	96,668	991,671
Non-Federal-aid:												
State highways	2,963,087	28,358,274	31,321,361	337,210	1,146,764	1,483,974	98,502	298,835	397,337	435,712	1,445,599	1,881,311
Local roads	16,025,291	122,248,701	138,273,992	1,591,683	3,829,322	5,421,005	129,648	554,325	683,973	1,721,331	4,383,647	6,104,978
Subtotal	18,988,378	150,606,975	169,595,353	1,928,893	4,976,086	6,904,979	228,150	853,160	1,081,310	2,157,043	5,829,246	7,986,289
All roads and streets:												
State highways	40,345,227	75,211,415	115,556,642	5,025,782	2,548,059	7,573,841	1,613,706	2,042,798	3,656,504	6,639,488	4,590,857	11,230,345
Local roads	20,636,655	122,576,323	143,212,978	2,021,438	3,829,553	5,850,991	445,583	554,325	999,908	2,467,021	4,383,878	6,850,899
TOTAL	60,981,882	197,787,738	258,769,620	7,047,220	6,377,612	13,424,832	2,059,289	2,597,123	4,656,412	9,106,509	8,974,735	18,081,244

[1] Includes 1,550 trucks of unknown type involved in traffic accidents of which 487 were involved in rural accidents and 1,063 were involved in municipal accidents.

to $34,900; and truck combinations, $39,500 to $83,400.

The comparison of involvement and cost rates in rural areas versus municipalities points to the fact that many of the accidents in cities were relatively minor events. For all classes of vehicles considered in the study, involvement rates ranged from 170 per 10 million vehicle-miles of travel in rural areas to 650 in municipalities, or a ratio of 1 to 3.8. Cost rates, on the other hand, ranged from $54,700 per 10 million vehicle-miles in rural areas to $109,500 in municipalities, a ratio of 1 to 2.

An analysis of the types of accidents shows that nearly one-half of all accidents in municipalities were collisions with parked vehicles and rear-end collisions. These two types of accidents accounted for only 15 percent of the total direct costs of accidents in municipalities. But regardless of the severity or costs of specific types of accidents, the fact still remains that a large part of the accident problem is concentrated in cities, and prevailing vehicle insurance rates for urban residents reflect that condition. Eighty-five percent of the accident involvements occurring in the State during the study year took place in municipalities, and those events accounted for 75 percent of the total direct costs of accidents.

A rather unusual finding of the study was the doubling of the accident cost rate for truck combinations in cities versus rural areas. A similar relationship did not hold for single-unit trucks. As shown in table 12, the cost of approximately 0.8 of a cent per vehicle-mile for combinations was uniform for all city size groups. A further analysis of these data indicated that the rates for combinations were

Table 16.—In-State travel of Illinois registered passenger cars and trucks, distributed by highway systems [1]

[thousands of vehicle-miles]

Highway systems	Travel of Illinois registered passenger cars			Travel of Illinois registered single-unit trucks			Travel of Illinois registered truck combinations			Travel of Illinois registered trucks of all types		
	Rural	Municipal	Total	Rural	Municipal	Total	Rural	Municipal	Total	Rural	Municipal	Total
Federal-aid primary and State highways	5,844,957	4,985,517	10,830,474	1,292,470	560,818	1,853,288	473,874	162,882	636,756	1,766,344	723,700	2,490,044
Federal-aid secondary highways:												
State highways	409,629	137,276	546,905	78,716	18,874	97,590	6,051	2,422	8,473	84,767	21,296	106,063
Local roads	1,066,522	133,976	1,200,498	223,545	14,263	237,808	5,413	647	6,060	228,958	14,910	243,868
Subtotal	1,476,151	271,252	1,747,403	302,261	33,137	335,398	11,464	3,069	14,533	313,725	36,206	349,931
Non-Federal-aid highways:												
State highways	586,552	1,403,184	1,989,736	126,168	155,565	281,733	24,595	33,061	157,656	150,763	188,626	339,389
Local roads	2,078,424	10,102,269	12,180,693	591,689	1,061,644	1,653,333	11,255	112,458	23,713	602,944	1,174,102	1,777,046
Subtotal	2,664,976	11,505,453	14,170,429	717,857	1,217,209	1,935,066	35,850	145,519	181,369	753,707	1,362,728	2,116,435
All roads and streets:												
State highways	6,841,138	6,525,977	13,367,115	1,497,354	735,257	2,232,611	504,520	198,365	702,885	2,001,874	933,622	2,935,496
Local roads	3,144,946	10,236,245	13,381,191	815,234	1,075,907	1,891,141	16,668	113,105	129,773	831,902	1,189,012	2,020,914
TOTAL	9,986,084	16,762,222	26,748,306	2,312,588	1,811,164	4,123,752	521,188	311,470	832,658	2,833,776	2,122,634	4,956,410

[1] Data source: *Motor Vehicle Use Study*, State of Illinois, Department of Public Works and Buildings, Division of Highways, October 1961.

Table 17.—Number of vehicles involved in traffic accidents and the direct costs of such accidents per 10 million vehicle-miles of travel, classified by major vehicle type and highway system

Highway systems	Illinois registered passenger cars			Illinois registered single-unit trucks			Illinois registered truck combinations			Illinois registered trucks, all types		
	Rural	Municipal	Total	Rural	Municipal	Total	Rural	Municipal	Total	Rural	Municipal	Total
NUMBER OF VEHICLES INVOLVED IN TRAFFIC ACCIDENTS PER 10 MILLION VEHICLE-MILES OF TRAVEL												
Federal-aid primary and State highways	152	445	287	84	312	153	65	323	131	79	314	147
Federal-aid secondary:												
State highways	485	432	472	100	(1)	149	(1)	(1)	(1)	97	(1)	144
Local roads	133	(1)	152	77	(1)	73	(1)		(1)	77	(1)	73
Subtotal	231	371	252	83	(1)	95	(1)	(1)	(1)	82	(1)	94
Non-Federal-aid:												
State highways	159	794	607	122	782	486	(1)	613	395	118	752	471
Local roads	283	776	692	138	531	390	(1)	645	617	141	542	406
Subtotal	256	778	680	135	563	404	(1)	638	546	137	571	416
All roads and streets:												
State highways	173	519	342	88	412	195	67	369	152	83	403	185
Local roads	232	770	643	121	524	359	(1)	642	591	123	535	366
TOTAL	191	672	493	100	479	266	73	468	220	95	477	258
DIRECT COST OF TRAFFIC ACCIDENTS PER 10 MILLION VEHICLE-MILES OF TRAVEL												
Federal-aid primary and State highways	$58,324	$91,431	$73,564	$35,157	$23,281	$31,563	$31,877	$107,021	$51,099	$34,277	$42,128	$36,559
Federal-aid secondary:												
State highways	80,372	92,529	83,424	18,379	(1)	24,626	(1)	(1)	(1)	17,615	(1)	23,170
Local roads	43,237	(1)	41,141	19,225	(1)	18,081	(1)		(1)	32,569	(1)	30,587
Subtotal	53,542	58,906	54,375	19,004	(1)	19,985	(1)	(1)	(1)	28,528	(1)	28,339
Non-Federal-aid:												
State highways	50,517	202,099	157,415	26,727	73,716	52,673	(1)	90,389	68,915	28,900	76,638	55,432
Local roads	77,103	121,011	113,519	26,901	36,070	32,788	(1)	49,292	55,287	28,549	37,336	34,355
Subtotal	71,252	130,901	119,683	26,870	40,881	35,683	(1)	58,629	59,619	28,619	42,776	37,735
All roads and streets:												
State highways	58,974	115,249	86,448	33,564	34,655	33,924	31,985	102,982	52,021	33,166	49,173	38,257
Local roads	65,618	119,747	107,026	24,796	35,594	30,939	(1)	49,010	77,051	29,655	36,870	33,900
TOTAL	61,067	117,996	96,742	30,473	35,213	32,555	39,511	83,383	55,922	32,136	42,281	36,481

[1] Sample was too small to provide significant data (20 or less sample cases).

Table 18.—In-State travel of Illinois registered passenger cars and trucks, distributed by highway systems and average daily travel per mile of road or street

Item of comparison	Federal-aid primary and State highways	Federal-aid secondary highways			Non-Federal-aid highways			All roads and streets		
		State highways	Local roads	Total	State highways	Local roads	Total	State highways	Local roads	Total
TRAVEL IN RURAL AREAS										
Miles of rural roads	8,625	1,618	10,050	11,668	2,391	79,503	81,894	12,634	89,553	102,187
Annual passenger car travel (1,000 vehicle-miles)	5,844,957	409,629	1,066,522	1,476,151	586,552	2,078,424	2,664,976	6,841,138	3,144,946	9,986,084
Average daily passenger car travel (1,000 v.-m.)	16,014	1,122	2,922	4,044	1,607	5,694	7,301	18,743	8,616	27,359
Average daily passenger car travel per mile of road	1,857	693	291	347	672	72	89	1,484	96	268
Annual truck travel (1,000 vehicle-miles)	1,766,344	84,767	228,958	313,725	150,763	602,944	753,707	2,001,874	831,902	2,833,776
Average daily truck travel (1,000 vehicle-miles)	4,839	233	627	860	413	1,652	2,065	5,485	2,279	7,764
Average daily truck travel per mile of road	561	144	62	74	173	21	25	434	25	76
TRAVEL IN MUNICIPALITIES										
Miles of streets	1,498	203	209	412	988	18,192	19,180	2,689	18,401	21,090
Annual passenger car travel (1,090 vehicle-miles)	4,985,517	137,276	133,976	271,252	1,403,184	10,102,269	11,505,453	6,525,977	10,236,245	16,762,222
Average daily passenger car travel (1,000 v.-m.)	13,659	376	367	743	3,844	27,678	31,522	17,879	28,045	45,924
Average daily passenger car travel per mile of street	9,118	1,852	1,756	1,803	3,891	1,521	1,643	6,649	1,524	2,178
Annual truck travel (1,000 vehicle-miles)	723,700	21,296	14,910	36,206	188,626	1,174,102	1,362,728	933,622	1,189,012	2,122,634
Average daily truck travel (1,000 vehicle-miles)	1,983	58	41	99	516	3,217	3,733	2,557	3,258	5,815
Average daily truck travel per mile of street	1,324	287	195	241	523	177	195	951	177	276
TOTAL TRAVEL										
Miles of roads and streets	10,123	1,821	10,259	12,080	3,379	97,695	101,074	15,323	107,954	123,277
Annual passenger car travel (1,000 vehicle-miles)	10,830,474	546,905	1,200,498	1,747,403	1,989,736	12,180,693	14,170,429	13,367,115	13,381,191	26,748,306
Average daily passenger car travel (1,000 v.-m.)	29,673	1,498	3,289	4,787	5,451	33,372	38,823	36,622	36,661	73,283
Average daily passenger car travel per mile of road and street	2,931	823	321	396	1,613	342	384	2,390	340	594
Annual truck travel (1,000 vehicle-miles)	2,490,044	106,063	243,868	349,931	339,389	1,777,046	2,116,435	2,935,496	2,020,914	4,956,410
Average daily truck travel (1,000 vehicle-miles)	6,822	291	668	959	929	4,869	5,798	8,042	5,537	13,579
Average daily truck travel per mile of road and street	674	160	65	79	275	50	57	525	51	110

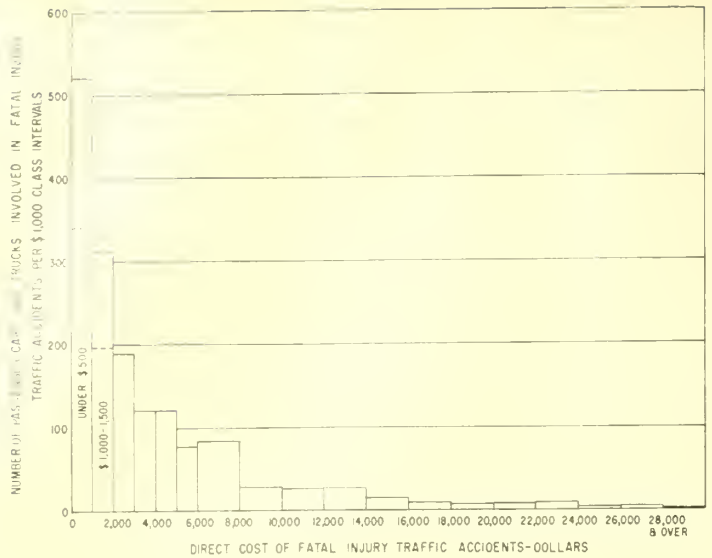

NUMBER OF PASSENGER CARS AND TRUCKS INVOLVED IN FATAL INJURY TRAFFIC ACCIDENTS PER $1,000 CLASS INTERVALS
UNDER $500
$1,000-1,500
0
100
200
300
400
500
600
0 2,000 4,000 6,000 8,000 10,000 12,000 14,000 16,000 18,000 20,000 22,000 24,000 26,000 28,000 & OVER
DIRECT COST OF FATAL INJURY TRAFFIC ACCIDENTS-DOLLARS

Figure 8.—Number of passenger cars and trucks (combined) involved in fatal injury traffic accidents, distributed according to direct costs.

influenced to a considerable extent by the occurrence of a limited number of fatal and nonfatal injury accidents in which the costs exceeded $10,000 per involvement. A summary of the number of municipalities and the population for each of the city size groups (given in tables 10—12) and total populations are shown in table 13.

The population group of 1,000,000 and over obviously applies to Chicago. Incorporated places surrounding the corporate area of Chicago such as Evanston, Oak Park, Berwyn, Cicero, and others were included in the lesser population groups. Forty-six percent of the accident involvements and 56 percent of the total costs of accidents occurring in municipalities of the State were traceable to the corporate area of Chicago. This finding was not unusual as 46 percent of the urban population of the State resided in the one city, and 45 percent of the Statewide municipal travel was performed there. In relating the costs of

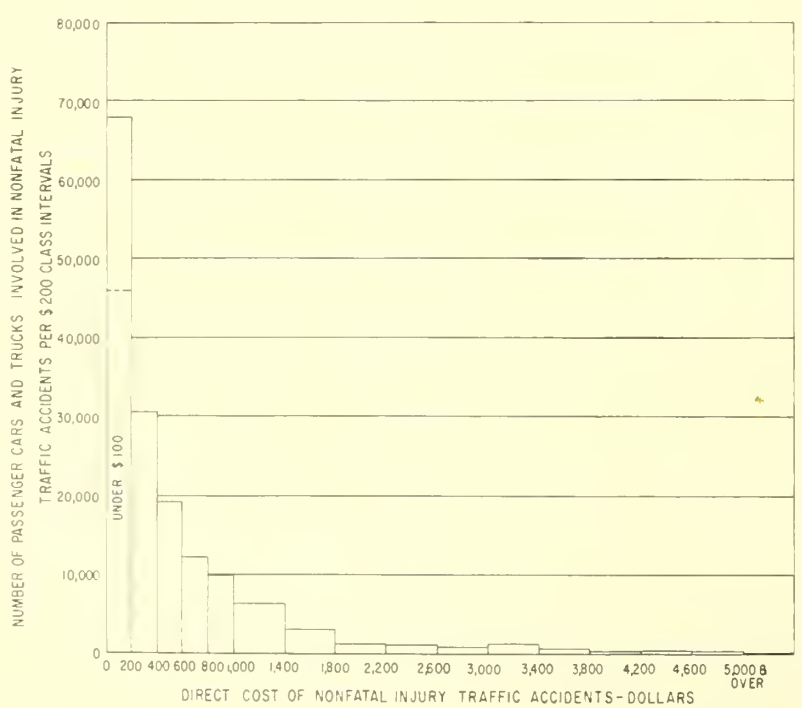

NUMBER OF PASSENGER CARS AND TRUCKS INVOLVED IN NONFATAL INJURY TRAFFIC ACCIDENTS PER $200 CLASS INTERVALS
UNDER $100
0
10,000
20,000
30,000
40,000
50,000
60,000
70,000
80,000
0 200 400 600 800 1,000 1,400 1,800 2,200 2,600 3,000 3,400 3,800 4,200 4,600 5,000 & OVER
DIRECT COST OF NONFATAL INJURY TRAFFIC ACCIDENTS-DOLLARS

Figure 9.—Number of passenger cars and trucks (combined) involved in nonfatal injury traffic accidents, distributed according to direct costs.

passenger car and truck accidents to travel o these vehicles in Chicago, the rate per vehicl mile was found to be 1.35 cents.

A recent publication of the Chicago Are Transportation Study (C.A.T.S.) provide useful comparisons of accident costs and rate for streets and highways of the Greate Chicago area (*3*). The area covered in th analysis included Cook and Du Page Counties the confines of which were nearly equivalen to the perimeters of the C.A.T.S. study.

The locations of traffic accidents occurrin in Cook and Du Page Counties during 195 were classified on the basis of three systems expressways, arterials, and local streets Accident rates and costs developed in th analysis are listed in table 14.

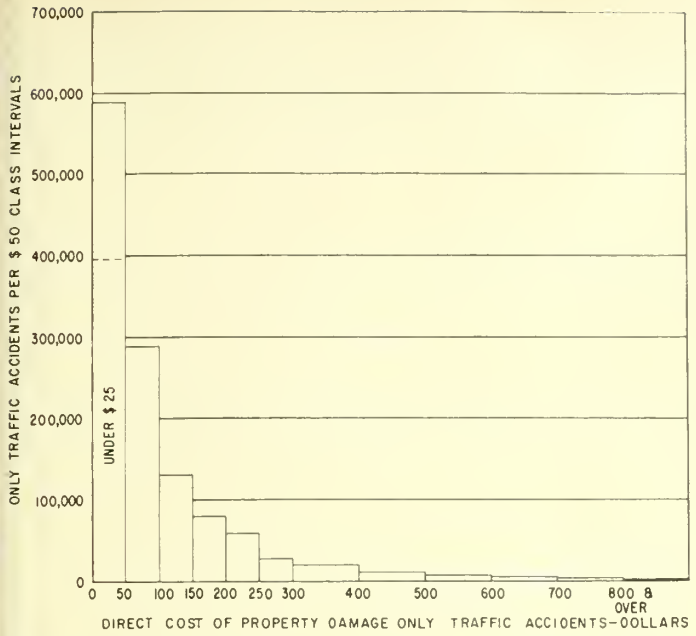
The cost of accidents per vehicle-mile o travel on all street systems of the two countie was calculated to be 1.32 cents, which wa slightly less than the rate of 1.35 cents for th corporate area of Chicago. Of primary in terest is the range in costs per vehicle-mil by street systems: Expressways, 0.31 of cent; arterials, 1.07 cents; and local streets 3.09 cents. Frequency rates were based o the number of accidents per 10 million vehicl miles rather than involvements, and thu direct comparisons cannot be made with th data shown in table 12. (In the C.A.T.S analysis, a conversion factor of 1.89 involve ments per traffic accident was used.) Result show that the chance of being involved in traffic accident on a local street was 20 time greater than on an expressway, and on arteria streets the accident rate was nearly 5 time that on expressways.

Frequencies and Costs of Accident Involvements Related to Highway Systems

Data included in tables 15 and 16 provide the necessary information to appraise the major highway systems of the State on the basis of accident frequencies and costs. The same limitations apply to this series of table as to tables 5–7. Sampling variability should be kept in mind when viewing the detailed information. Values shown for subtotals an totals obviously are supported by a greate number of sample cases than the componen values that make up the totals. Table cell believed to have too few sample cases t provide significant comparisons are indicate by footnote in table 17. No estimates o sampling error have been computed, however

Table 19.—Average daily travel of Illinois passenger cars and trucks on Illinois roads and streets, 1958

Highway system	Average daily traffic per mile of road or street	
	Rural	Municipal
Federal-aid primary	2,418	10,442
Federal-aid secondary	421	2,044
Non-Federal-aid	114	1,838
All systems	344	2,454



gure 10.—Number of passenger cars and trucks (combined) involved in property damage only traffic accidents, distributed according to direct costs.

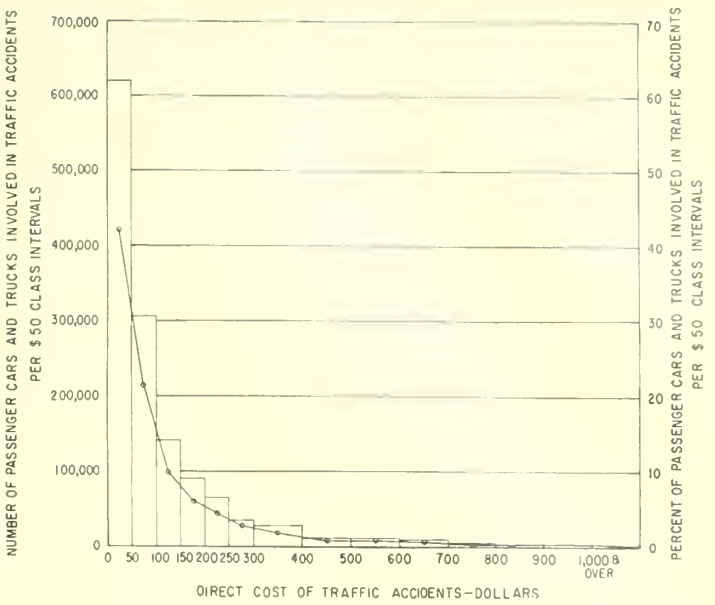


Figure 11.—Number and percent of passenger cars and trucks (combined) involved in traffic accidents, distributed according to direct costs.

Accident involvement and cost rates, as ported in table 17, point to the fact that ssenger car drivers traveling on local rural ads and on city streets (principally of the sidential class) experienced more accidents a vehicle-mile basis than when driving on ate highways. Rates on rural State high-ays were 173 involvements per 10 million hicle-miles as compared to 232 involvements local roads, or a ratio of 1 accident volvement on State highways for every 1.3 volvements on local roads. In municipali-es, the rates per 10 million vehicle-miles ere 519 and 770, respectively, or a ratio of to 1.5. Costs per vehicle-mile for passenger r involvements ranged from 0.59 of a cent rural State highways to 0.66 of a cent on cal rural roads. A similar comparison for unicipalities indicated costs of 1.15 cents id 1.20 cents. Involvement ratios were mewhat greater in the State–local compari-ns than were the cost ratios, which indicates at accidents on the local systems tended to less severe or costly. Involvement rates r trucks of all types were higher on local ads and streets than on State highways, it costs per vehicle-mile indicated an in-rse relation.

A comparison of involvement and cost rates the basis of the three classes of highways—ederal-aid primary, Federal-aid secondary, id non-Federal-aid—is not too conclusive. owever, the emphasis placed on improving ie design of major highways shows some nefits from the standpoint of accident fre-iencies and costs. One principal observa-on is that the roads and streets not a part of ie Federal-aid systems should not be over-oked in accident reduction programs. This ass of roads and streets, composed largely of unty and township roads in rural areas and sidential streets in municipalities, is repre-sentative of 82 percent of the road and street mileage of the State. During the year of the study, these facilities accounted for 51 percent of the travel, 73 percent of the accident involvements, and 64 percent of the total direct costs of accidents.

The percentage distribution of travel, accident involvements, and accident costs is illustrated in figure 12 for the three classes of highways. The system classifications used in the study are fairly realistic from the standpoint of vehicle usage, particularly in rural areas. A preferred classification for major cities would be expressways, arterials, and residential streets. Streets of the Federal-aid secondary classification represent a very small part of the total municipal mileage, as shown in table 18.

Average daily travel of Illinois passenger cars and trucks on the three systems during 1958 was distributed as shown in table 19.

REFERENCES

(*1*) *Cost of Motor Vehicle Accidents to Illinois Motorists, 1958,* State of Illinois Department of Public Works and Buildings, Division of Highways, December 1962.

(*2*) *Motor Vehicle Use Study,* State of Illinois Department of Public Works and Buildings, Division of Highways, October 1961. (Study period covered 12-month interval beginning in October 1957.)

(*3*) *Accident Costs and Rates on Chicago Area Streets and Highways,* by D. P. Jorgenson, Chicago Area Transportation Study Research News, vol. 4, No. 4, March 30, 1962, pp. 2–11.

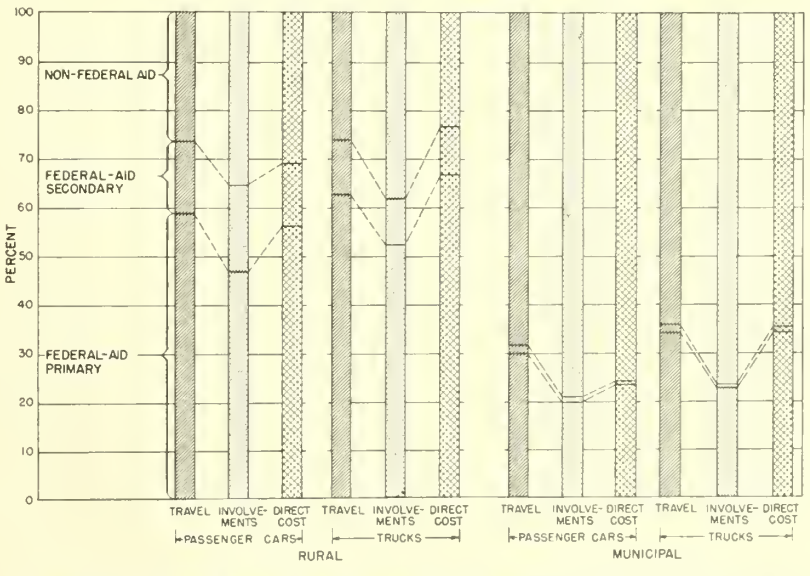


Figure 12.—Percentage distribution of travel, accident involvements, and accident costs on the basis of major vehicle type, rural and municipal location, and highway system.

Silicones as Admixtures for Concrete

BY THE
MATERIALS RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

Reported by [1] WILLIAM E. GRIEB
Highway Research Enginee

This article presents a report on the results obtained from tests in which eight different silicones were used as admixtures for portland cement concrete. These tests were made because previous tests had shown the addition of silicone to be beneficial in preventing scaling caused by de-icing agents.

The results of tests from the lastest study showed that some of the silicones improved the strength and durability of the concrete. An optimum amount of silicone admixture was required to obtain maximum strength and durability. However, in most of the tests, the addition of silicones retarded the setting time of the concretes more than can usually be tolerated for normal construction purposes.

Introduction

A RECENT REPORT of the Bureau of Public Roads [2] showed that given amounts of a certain silicone used as an admixture for concrete were effective in preventing scaling caused by de-icing agents. This silicone also increased the compressive strength of the concrete but caused a marked retardation in the setting of the concrete. Because of these effects on concrete, additional tests were made to determine whether other silicones used as admixtures would have similar effects on concrete.

Eight different silicones, manufactured by the three major producers, were used in this project. Tests were made to determine the effect of silicone admixtures on the properties of fresh concrete and on the strength and durability of hardened concrete. Some of the tests were limited because sufficient quantities of the silicone samples were not available.

[1] Presented at the 42d annual meeting of the Highway Research Board, Washington, D.C., January 1963.

[2] *Resistance of Concrete Surfaces to Scaling by De-Icing Agents*, by W. E. Grieb, George Werner, and D. O. Woolf, PUBLIC ROADS, vol. 32, No. 3, Aug. 1962, pp. 64—73.

Conclusions

Based on results of tests in which only one brand of cement was used, the following conclusions are warranted. These conclusions apply specifically to concrete prepared with the materials, mixes, and mixing procedures described in this article.

- When used as admixtures in certain amounts, solutions of sodium methyl siliconates increased the compressive strength and durability of concrete.
- The alkyl silane esters and the silicone resin emulsion types of silicones in most cases either had no effect or were detrimental to the compressive strength and durability of concrete.
- There appears to be a critical amount of silicone admixture needed to obtain maximum compressive strength or durability of the concrete. This amount varies according to the properties of the silicones.
- In most cases, silicones retarded the setting time of the concrete. When the silicones were used in amounts needed to obtain maximum strength or durability of concrete, most of them caused retardation of the set greater than can be tolerated for normal construction purposes.
- The use of silicones as admixtures had no appreciable effect on the water required for a given slump or on the air content of the concrete.

Materials

The tests were made on air-entrained concrete prepared with varying amounts of eight different silicone solutions. The physical and chemical properties of these silicone solutions are given in table 1. The silicones are grouped into three general classes. Four of them (silicones *A*, *B*, *C*, and *D*) are classified as sodium methyl siliconates, two (silicones *E* and *F*) are classified as alkyl silane esters, and two (silicones *G* and *H*) are classified as silicone resin emulsions. Typical infrared spectra of the silicones are given in figure 1. The silicones in each group have the same general characteristic spectra.

With the exception of the two milky whit emulsions of silicones *G* and *H*, all silicone were colorless liquids. The solvent or thinn for the six silicones *A*, *B*, *C*, *D*, *G*, and *H* wa water and for the other two (*E* and *F*) was a alcohol. Except for the silicone admixture the same concrete materials were used for a of the tests. The cement was a type I port land cement having an equivalent alka content of 0.6 of a percent. The chemic analysis of the cement is given in table The aggregates were similar to those use in the previous investigation of a silicone a an admixture. These were a siliceous san having a fineness modulus of 2.75 and uniformly graded crushed limestone havin a 1-inch maximum size. A commerciall available aqueous solution of neutralize Vinsol resin was used to entrain air.

Mix Data

The mix data for the concretes are give in table 3. The concrete contained 6 bags cement per cubic yard. The air content wa approximately 5 percent and the slump wa about 3 inches. A control or reference mi was made each day without silicone, and th mixes containing silicone were compared the corresponding control mix made on th same day. The average properties for all the control mixes are given in footnote 1 table 3.

The total solids in the silicone solution added to the mixes ranged from 0.01 percen to 1.33 percent by weight of the cement. Th concentration of the total solids in the eigh silicone solutions differed. For convenienc in designing the mixes, data from literatu furnished by the producers, which gives th approximate percentage of total solids i each solution, were used. These data a given in footnote 2 of table 3.

The actual percentage of total solids i six of the eight solutions was determine chemically; these percentages are shown i table 1. These results were within 5 percen age points of the amounts used for designin the mixes. For silicones *E* and *F*, the alky silane esters, it was impossible to determi

Table 1.—Physical and chemical properties of silicone solutions

	Sodium methyl siliconate				Alkyl silane ester		Silicone resin emulsion	
	Sodium methyl siliconate (Sodium salt of methyl polysiloxane) [1]				Methyl chlorosilane M ester	Ethyltriethoxysilane [1]	Silicone resin emulsion of silicone resin [1]	
							Nonionic	Anionic
	A	*B*	*C*	*D*	*E*	*F*	*G*	*H*
Physical properties:								
pH (electrometric method)	12.1	12.0	12.2	12+	2.6	7.2	7.2	8.4
Specific gravity $\frac{25°}{4°}$C	1.244	1.252	1.102	1.227	0.952	0.901	1.027	1.008
Chemical analysis:								
Total solids (nonvolatile at 150° C., 90 min.) Percent	33.5	33.3	32.1	30.1	([3])	([3])	41.4	16.9
Total sodium as Na_2O do	10.4	10.3	11.2	12.4				
Silicon (Si) do	8.2	8.1	8.5	5.6	21.7	3.8	8.9	3.4
Chlorine (Cl) do					[2] 0			
Silicone solids as $CH_3SiO_{1.5}$ do	19.6	19.4	20.3	13.4				
Molecular ratio ($CH_3SiO_{1.5}$ to Na_2O) do	1.7	1.7	1.7	1.0				
Infrared analysis of active constituent	All four materials were similar and showed a methyl siliconate structure. Silicone *D* had a greater amount of sodium carbonate impurity than the others.				Spectra of both materials were fairly similar; they had an alkyl silane ester structure. *F* had ethyl groups, and *E* had mostly methyl substitution.		Both materials had similar spectra of presumably condensed silicones having ethyl substitution.	
Infrared analysis of volatile solvent or thinner					Both solvents appeared to be an alcohol type, but exact identification was difficult because of some volatility of the active constituent.			
Probable formulas	$[CH_3Si(OH)_2O^-]$ Na^+(in dilute aqueous solution) $[CH_3SiO_2Na]_n$ (in dry form)				$(CH_3)_nSi(OCH_3)_{4n}$	$C_2H_5Si(OC_2H_5)_3$	$\left[R''O(R'_x)SiO_{\frac{4-x}{2}}\right]R'$	

[1] Producer's description.
[2] Qualitative test.
[3] Not determined because of volatility of silicone material.

he amount of total solids because of the olatility of the silicone materials.

For the six silicones *A*, *B*, *C*, *D*, *E*, and *F*, he assumed concentration of the solutions as 30 percent total solids. For this concenation, 10 ounces of the solution per bag of ement is equivalent to 0.2 of a percent of otal solids by weight of cement. In table 3, he amount of the silicone solution used each mix is given as the weight of total olids in the quantity of solution used, exressed as a percentage of the weight of ement. It is also given as the number of nces of the solutions per bag of cement.

Mixing and Curing

The mixing and curing was completed in ccordance with standard laboratory proedures. The aqueous solution of each sili one was added, with part of the mixing water, o the cement and aggregates in the mixer, rior to the addition of the aqueous solution the air-entraining admixture.

ASTM standard methods were followed in aking tests on the plastic concrete and in olding, curing, and testing the specimens of ardened concrete. The tests for outdoor caling were made as described in reference *2*.

Water and Air Content

Data showing the effect of silicones as adixtures on the water and air content of conrete are presented in table 3 and figure 2. s shown in figure 2, concretes made with licone admixtures generally needed less ater than the control concrete to provide the same slump. This reduction in water requirement however was 3 percent or less for most of the mixtures. In 9 of the 11 mixes that did show a reduction of more than 3 percent, more than 10 ounces of the silicone solution per bag of cement (0.2 of a percent of total solids by weight of cement) had been used. The erratic data in amount of mixing water obtained from use of some of the silicones may be attributed to time and mold limitations. Because of these limitations not all of the mixtures prepared with the different amounts of any specific silicone were made on the same day. Mixtures containing varying amounts of the same silicone were therefore compared with different control mixes.

Table 2.—Chemical analysis and physical properties of portland cement

Chemical composition:	
Silicon dioxide percent	20.9
Aluminum oxide do	6.0
Ferric oxide do	2.5
Calcium oxide do	65.3
Magnesium oxide do	1.4
Sulfur trioxide do	2.2
Loss on ignition do	0.7
Insoluble residue do	0.19
Sodium oxide do	0.14
Potassium oxide do	0.75
Chloroform soluble do	0.007
Free lime do	0.76
Equivalent alkali as Na_2O do	0.63
Computed compound composition:	
Tricalcium silicate percent	57
Dicalcium silicate do	17
Tricalcium aluminate do	12
Physical properties:	
Apparent specific gravity	3.14
Specific surface (Blaine) sq. cm./g	3,250
Autoclave expansion percent	0.05
Normal consistency do	24.2
Time of setting (Gillmore test)	
Initial hours	4.25
Final do	6.83
Compressive strength (1:2.75 mortar):	
3 days p.s.i.	2,850
7 days do	3,830
28 days do	5,170
Mortar air content percent	9.4

For mixtures containing silicones *B* and *E*, except for one mixture for each, a progressive reduction occurred in the amount of mixing water required as the amount of silicone was increased. Silicone *H* also caused a reduction in the amount of mixing water required when 0.5 of a percent or more of silicone solids was used. Addition of silicone *C* or *F* in an amount up to 0.5 of a percent of silicone solids reduced the mixing water requirements; when more *C* or *F* was used the mixing water requirement increased.

Although the general trend was for greater reductions in the water requirements as the amount of silicone used was increased, these data fail to show that the silicones used are effective water-reducing agents.

The use of silicones as admixtures had some effect on the air content of the concrete. Table 3 gives the amount of air-entraining agent needed in the mixes prepared with the silicone admixtures as a percentage of the amount of agent needed in the control concrete made on the same day. The same data are also shown in the upper portion of figure 3. In general, when mixes were prepared with less than 0.2 of a percent of total silicone solids, less air-entraining agent was needed than had been used in the control mix. However, for mixes prepared with larger amounts of the silicones, more air-entraining agent was required than for the control mix. With one exception, more air-entraining agent was required for all mixes prepared with silicone *D*.

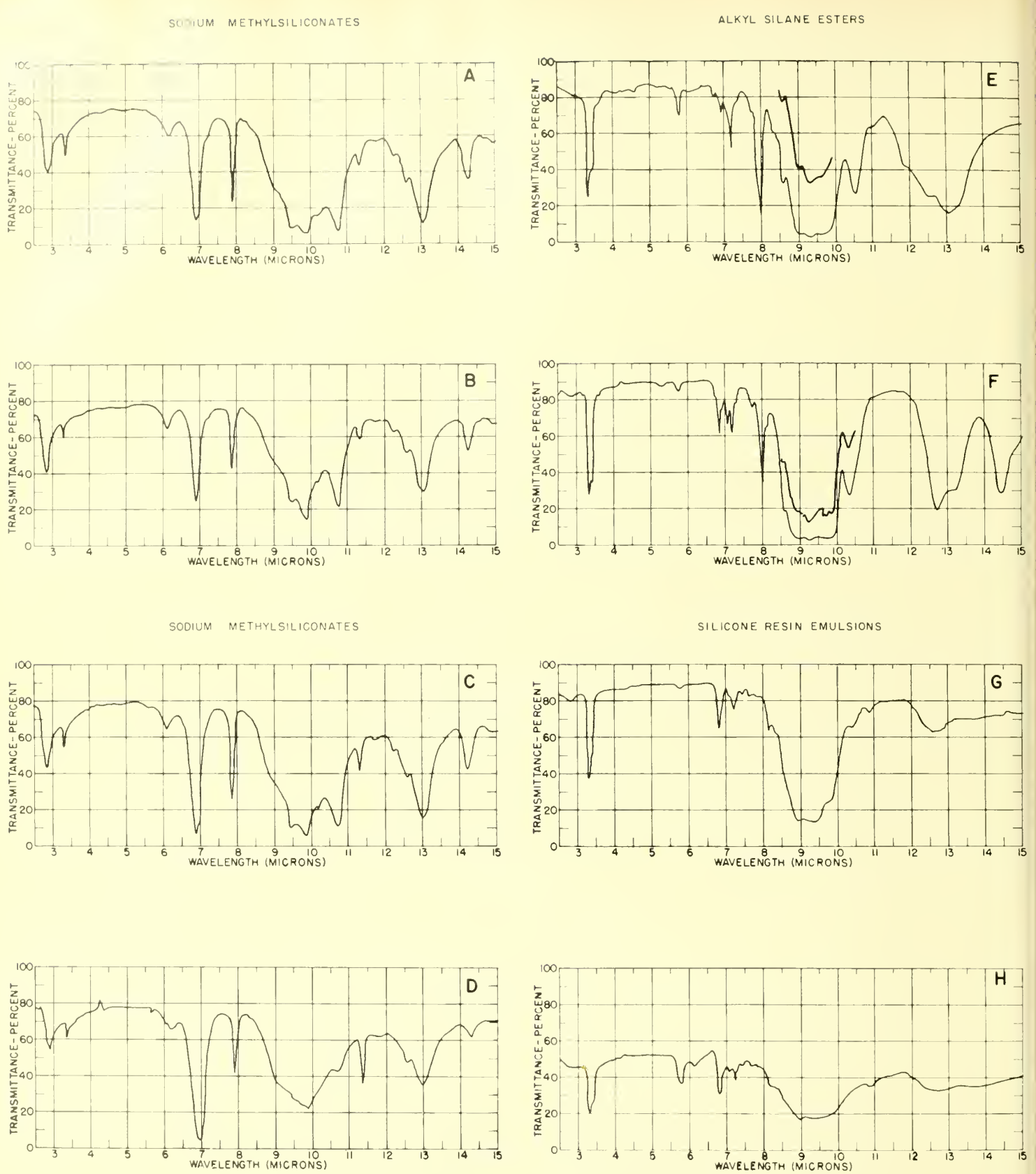


Figure 1.—Infrared spectra of silicone admixtures.

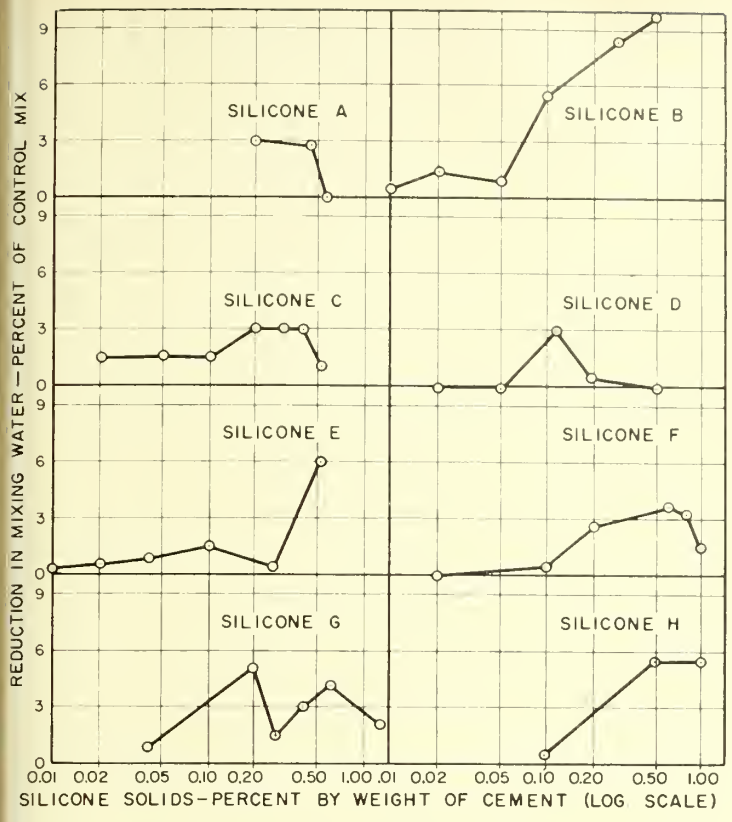


Figure 2.—Effect of silicone on reduction in amount of mixing water—based on control mix.

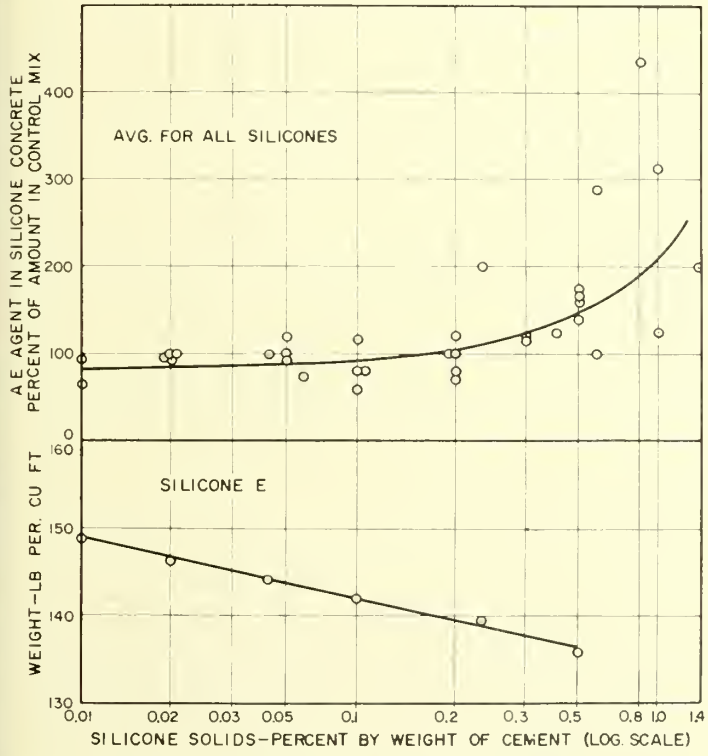

AVG. FOR ALL SILICONES
AE AGENT IN SILICONE CONCRETE
PERCENT OF AMOUNT IN CONTROL MIX
SILICONE E
WEIGHT—LB PER CU FT
0.01 0.02 0.03 0.05 0.1 0.2 0.3 0.5 0.8 1.0 1.4
SILICONE SOLIDS—PERCENT BY WEIGHT OF CEMENT (LOG. SCALE)

Figure 3.—Effect of silicones on amount of AE agent needed and of one silicone on unit weight of concrete.

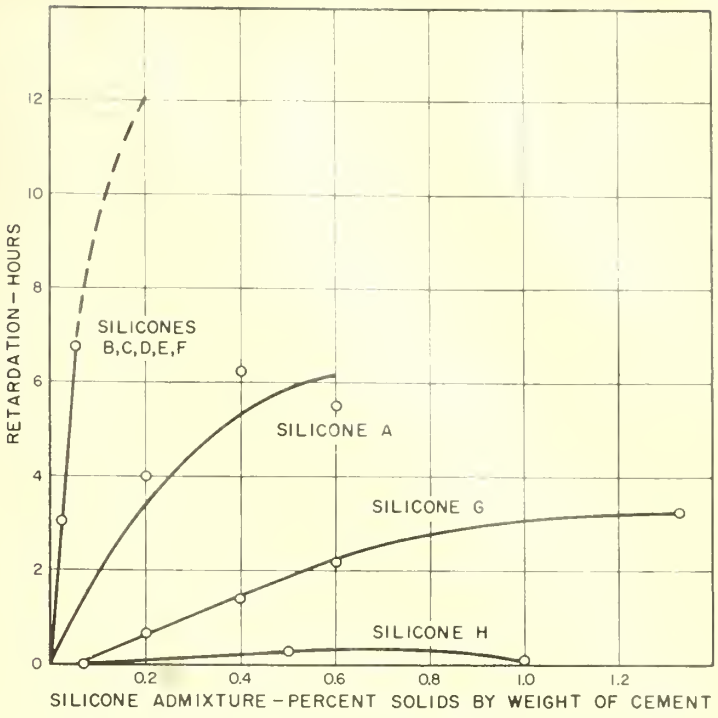


Figure 4.—Relation between amount of silicone added and retardation.

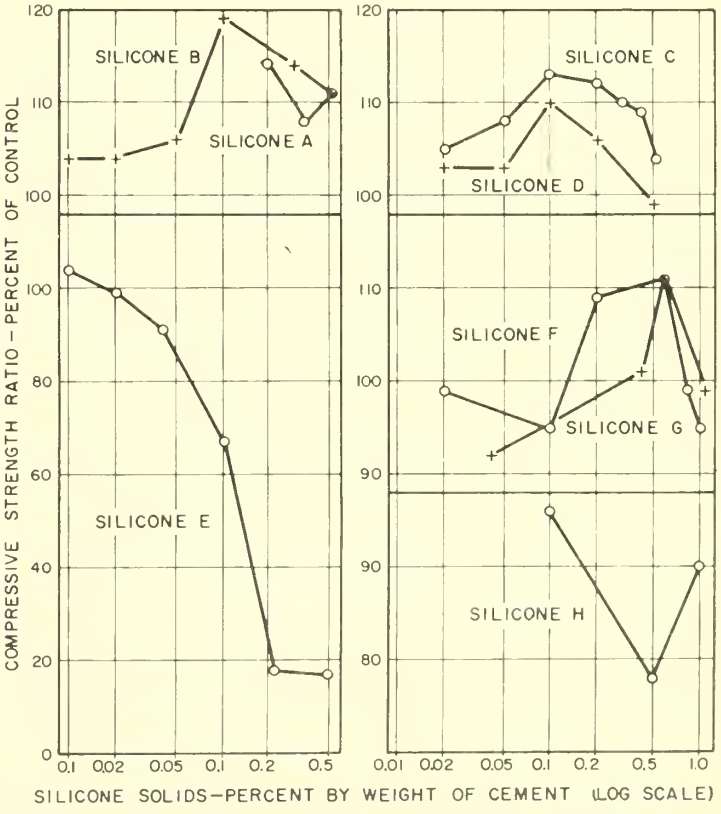


Figure 5.—Effect of silicones on compressive strength at 28 days.

Table 3.—Mix data [1]

Silicone, total solids	Liquid	Slump	Reduction in water	Air	A.E.A.[4]	Weight hardened concrete[5]
Percent by wt. of cement	*Oz./bag of cement*	*In.*	*Pct.*	*Pct.*	*Pct.*	*Lb./cu. ft.*
A:						
0.20	10.0	3.1	2.9	5.3	100	
.40	20.0	3.3	2.9	5.7	100	
.60	30.0	2.5	0	4.9	75	
B:						
0.01	0.5	2.9	0.5	5.3	94	
.02	1.0	3.1	1.4	5.5	94	
.05	2.5	3.2	0.9	4.7	100	
.10	5.0	3.0	5.4	4.9	80	
.30	15.0	3.1	8.4	5.0	120	
.50	25.0	3.3	9.9	5.5	160	
C:						
0.02	1.0	3.2	1.6	5.0	93	
.05	2.5	3.2	1.6	5.2	93	
.10	5.0	3.2	1.5	5.0	80	
.20	10.0	2.6	3.0	5.0	100	
.30	15.0	2.9	3.0	5.0	117	
.40	20.0	2.5	3.0	5.5	125	
.50	25.0	3.0	1.1	5.1	174	
D:						
0.02	1.0	2.5	0	5.0	100	
.05	2.5	3.0	0	5.4	120	
.10	5.0	2.7	3.0	5.5	117	
.20	10.0	2.8	0.5	5.1	120	
.50	25.0	2.8	0	5.4	140	
E:						
0.01	0.5	2.9	0.4	5.9	65	148.7
.02	1.0	3.0	0.6	6.0	70	146.4
.04	2.0	3.0	0.9	6.8	80	144.2
.10	5.0	3.7	1.5	8.0	188	142.0
.25	12.5	4.2	0.4	4.7	200	139.5
.50	25.0	2.7	6.0	4.5	167	135.9
F:						
0.02	1.0	3.3	0	7.2	70	
.10	5.0	3.2	0.5	6.8	70	
.20	10.0	3.0	2.5	6.0	80	
.60	30.0	3.5	3.8	4.2	287	
.80	40.0	3.0	3.3	4.5	437	
1.00	50.0	3.5	1.5	4.5	313	
G:						
0.04	1.5	3.2	0.9	6.3	50	
.20	7.5	3.4	5.1	9±	0	
.27	10.0	4.7	1.5	9±	80	
.40	15.0	4.2	3.1	8.0	200	
.60	20.0	2.9	4.2	5.1	100	
1.33	50.0	2.5	2.1	5.1	200	
H:						
0.10	10.0	2.5	0.6	5.0	60	
.50	50.0	3.0	5.4	8.5	187	
1.00	100.0	3.0	5.4	5.0	125	

[1] Control mix (average properties):
Proportions, 94–200–300.
Cement, 6.0 bags per cubic yard.
Slump, 3.0 inches.
Water, 5.58 gal. per bag.
Air-entraining agent, 20.7 ml. per bag.
Weight of hardened concrete, 149.1 lb. per cu. ft.
Air content, 5.2 percent.

[2] Based on total solids for each silicone, from information furnished by the producers, 30 percent solids for silicones *A*, *B*, *C*, *D*, *E*, and *F*, 40 percent for silicone *G* and 15 percent for silicone *H*.

[3] Reduction in water as compared with that required for the control mix made on the same day.

[4] Relative amount of air-entraining agent used, amount used in control mix considered 100 percent.

[5] Weight determined on cylinders prior to testing for compressive strength.

When silicone *E* was used, the concrete expanded during the hardening process; when the largest amount of silicone *E* (0.5 of a percent of solids by weight of cement) was used the concrete expanded 1 inch above the tops of the 6- by 12-inch cylinder molds. The air content of this plastic concrete, determined immediately after its mixing, was 4.5 percent. The unit weight of the hardened concrete for each of the mixes prepared with silicone *E* was determined on the cylinders prior to their being tested for compressive strength. These weights are shown in table 3 and the lower portion of figure 3. The weight of the control concrete was 149.1 pounds per cubic foot, whereas the weight of the concrete prepared with 0.5 percent silicone solids was only 135.9 pounds per cubic foot. As the weights of the two plastic concretes immediately after mixing were nearly the same, the concrete containing 0.5 of a percent percent of silicone solids expanded about 10 percent.

Tests were made to determine the cause of the expansion of the concrete prepared with silicone *E*. It was found that when a silicone *E* solution is treated with saturated limewater, it hydrolyzes and produces a mixture of alcohol containing perhaps both the methyl and ethyl types. As the parent silicone is an ester, such hydrolysis would be expected. The same result could be expected when silicone *E* is added to concrete where lime is immediately produced by the reaction of cement with mixing water. If the alcohols are produced in a gaseous form, this would account for the foaming (swelling) observed.

Retardation of Setting Time

The effect of different amounts of the silicone solutions on the retardation of the setting time of the concrete was determined by use of the Proctor penetration test (ASTM C 403). This test was made as described in reference *3*.[3] Retardation is the difference in time required for concrete prepared with the silicone admixtures and the control concrete made on the same day to support penetration loads of 500 p.s.i. The results of these tests are shown in table 4. Readings were taken at regular intervals for about 15 hours or until about 11 p.m. If the test specimens had not reached a penetration load of 500 p.s.i. by that time, the readings were resumed the next morning, but the concrete usually hardened before then.

The results of these tests for a penetration load of 500 p.s.i. are shown in figure 4. When silicones *B*, *C*, *D*, *E*, or *F* were used in amounts of only 0.05 of a percent of silicone solids, the retardation was approximately 6 hours. When 0.2 of a percent of silicone solids was used, the retardation was probably about 12 hours. It is estimated that a further increase in the amount of silicone used would cause only a small increase in the retardation. It was estimated that if 0.5 of a percent of solids were to be used the retardation would be between 15–20 hours. These five silicones are considered to retard the setting of the concrete more than would be desirable for normal construction purposes.

The use of 0.2 of a percent of solids of silicones *A* and *G* retarded the setting of the concrete 4 hours and three-fourths of an hour respectively, based on a 500 p.s.i. load in the Proctor test. Silicone *H* had no appreciable effect on the retardation of the concrete.

[3] *Water-Reducing Retarders for Concrete*, by W. E. Grieb, G. Werner, and D. O. Woolf, PUBLIC ROADS, vol. 31, No. 6, Feb. 1961, pp. 136–152.

Table 4.—Results of retardation and strength tests

Silicone, total solids	Air	Proctor penetration test, retardation at 500 p.s.i.[1]	Crushing[2] strength at—	
			7 days	28 days
Percent by wt. of cement	*Percent*	*Hr., Min.*	*Percent*	*Percent*
A:				
0.20	5.3	4:15	107	114
.40	5.7	6:15	104	108
.60	4.9	6:30	100	111
B:				
0.01	5.3	1:30	104	104
.02	5.5	2:30	105	104
.05	4.7	6:45	112	106
.10	4.9	[4] 12	116	119
.30	5.0	----------	110	114
.50	5.5	----------	107	111
C:				
0.02	5.0	2:35	107	105
.05	5.2	6:35	108	108
.10	5.0	[4] 11:30	108	113
.20	5.0	----------	113	112
.30	5.0	----------	109	110
.40	5.5	----------	109	109
.50	5.1	[4] 15	106	104
D:				
0.02	5.0	1:10	100	103
.05	5.4	5:00	102	103
.10	5.5	9:45	112	110
.20	5.1	----------	105	106
.50	5.4	----------	100	99
E:				
0.01	5.9	2:20	106	104
.02	6.0	5:15	102	99
.04	6.8	8:40	100	91
.10	8.0	----------	65	67
.25	4.7	----------	21	18
.50	4.5	----------	18	17
F:				
0.02	7.2	3:45	95	99
.10	6.8	[4] 10	97	95
.20	6.0	[4] 11	114	109
.60	4.2	[4] 12	113	111
.80	4.5	----------	102	99
1.00	4.5	----------	93	95
G:				
0.04	6.3	0:35	95	92
.20	[3] 9±	0:40	77	79
.27	[3] 9±	1:35	72	69
.40	8.0	1:40	102	101
.60	5.1	2:10	107	111
1.33	5.1	3:15	98	99
H:				
0.10	5.0	0	100	96
.50	8.5	0:20	79	78
1.00	5.0	0:05	90	90

[1] Retardation is delay in time of hardening of concrete containing silicones as compared with control concrete made on the same days. Average time for control concrete to reach Proctor penetration load of 500 p.s.i. was 4 hrs. 15 min., and for 4,000 p.s.i. was 7 hrs., 20 min.

[2] Strength expressed as ratio (in percent) of the strength of the concrete containing silicones to the strength of the control concrete made on the same day. Average strength of control concrete was 4,140 p.s.i. at 7 days and 5,220 p.s.i. at 28 days.

[3] Air content high, strength values disregarded.

[4] Time estimated.

Strength Tests

Compressive strength tests were made at ages of 7 and 28 days on concrete prepared with different amounts of the silicone admixtures. These strengths were compared with the strengths of the control concrete made and tested on the same days. In table 4, the strength of the concrete prepared with silicone admixtures is given as the percentage of that of the corresponding control concrete. The relative compressive strengths at 28 days are shown in figure 5.

Concretes prepared with silicones *A*, *B*, *C* and *D* (the sodium methyl siliconates) regardless of the amount used had higher strength than the control concrete, except for one mixture.

When silicone *E* was used in amounts of ess than 0.02 of a percent of solids, the strengths of these concretes were slightly higher than those of the control mix. When amounts of silicone solid greater than 0.02 of a percent were used, the strengths decreased considerably as the amount of silicone used was increased. When 0.50 of a percent of solids was used, the strength of concrete containing silicone *E* was only 18 percent of that of the corresponding control mix. This loss in strength was related to the foaming of the concrete previously mentioned.

For several of the mixes containing silicones *F* and *G*, the strengths were lower than that of the control concrete. However an examination of the data shows that these mixes contained 6.0 percent or more air.

Only three different amounts of silicone *H* were used. When 0.50 of a percent of solids of this material was used, a reduction in strength of 21 percent was obtained, but this mix had an air content of 8.5 percent. The other two mixes containing silicone *H* both showed slight reductions in strength. With the exception of silicones *E* and *H*, an optimum amount appears to exist at which the other silicones provide the maximum strength.

Laboratory Freezing and Thawing

Laboratory freezing and thawing tests were made on some of the mixes included in the strength tests. These tests were made on 3- by 4- by 16-inch beams, which were frozen in air and thawed in water in accordance with ASTM Method C 291. These tests were continued through 1,000 cycles of freezing and thawing; at 300 cycles only one of the mixes showed a loss in N^2 of more than 10 percent. Table 5 gives the durability factors of the concretes prepared with the different silicones at 1,000 cycles and the durability factor of the control mix. In addition, the relative durability factor is also given for each mix. This is the ratio of the durability factor of the silicone concrete to the durability factor of the control mix. A relative durability of 80 percent or more for concrete prepared with admixtures is acceptable. This durability is specified in AASHO Specification M 154 for air-entraining admixtures and is contained in the proposed specification for retarders made by the Subcommittee III–h of ASTM Committee C–9 (ASTM Designation C 494–62 T). On the basis of durability, all of the silicones used are acceptable. Although there appears to be an optimum amount of silicone admixture for obtaining maximum durability, these tests were too limited to determine this quantity.

Outdoor Scaling Tests

Outdoor exposure tests were made on 16- by 24- by 4-inch slabs to determine the effect silicone admixtures have on the resistance of concrete to scaling caused by de-icing agents. A description of the test is given in reference 2, results also are given for tests in which a silicone similar to silicone *A* was used. Those tests showed that the use of silicone in proper amounts was effective in preventing scaling.

Similar exposure tests were made on concretes in which silicones *B* and *C* had been used. At the time this article was prepared these specimens had been exposed for only one winter. At the last inspection neither the control slabs nor the slabs containing silicone showed any appreciable scaling. All slabs were given a rating of less than 2. These tests are being continued.

Summary

Use of the four silicones classified as sodium methyl siliconates—silicones *A*, *B*, *C*, and *D*—gave the best results. These were all furnished in about the same concentration, about 30 percent solids. Three of these silicones, *B*, *C*, and *D*, retarded the setting time of the concrete much more than would be desirable for ordinary usage.

From the available data, if these three silicones had been used in amounts of 0.2 of a percent of silicone solids by weight of the cement, the retardation of set would have been more than 10 hours. Use of this same amount of silicone *A* caused retardation of only 4 hours. Concretes having 10 to 20 percent higher strength than the control mixes were obtained from mixtures prepared with each of these four silicones. The most favorable results were obtained by use of 0.1 to 0.2 of a percent of silicone solids. Freezing and thawing tests in the laboratory showed concretes prepared with silicones *A*, *B*, and *C* to have practically the same or greater durability than the control concrete. Tests for durability were not made on concretes prepared with silicone *D* because of the lack of material.

The two silicones classified as alkyl silane esters, silicones *E* and *F* were unstable. It was not possible to determine the amount of total solids in these solutions because of the volatility of the silicone materials. These two silicones used as an admixture caused excessive retardation of the setting time of the concrete. Silicone *E* caused a reduction in the strength of concrete by foaming during hardening. There was a corresponding reduction in the weight of the hardened concrete. Concrete prepared with silicone *F* had strengths 10 to 15 percent greater than that of the control concrete when 0.2 to 0.6 of a percent of solids were used. There is no apparent reason for the differences in the behavior of these two similar materials. Concrete prepared with either of these materials had good durability but only a few mixes were tested and these all contained more air than the control concrete.

The use of silicones *G* and *H*, which were classified as silicone resin emulsions, had beneficial effects on the properties of the concrete only in isolated cases. Their use provided unpredictable results on reduction in mixing water and air content. It appears that if either were to be used in construction, very careful control of the amount of silicone would be required. Silicone *G* caused only a modest amount of retardation of setting time of concrete, however silicone *H* had practically no effect. When used in amounts that did not cause excessive amounts of entrained air, concretes containing each of these silicones had 90 to 109 percent of the strength of the control concrete. Only one concrete prepared with silicone *G* was tested for resistance to freezing and thawing. Although this concrete had low strength, its air content was high and the relative durability was almost equal to that for the control concrete.

The retardation of the setting time offers a problem that must be resolved before this material can be used commercially. However, the tests reported here show that when some of these silicones are used as admixtures in concrete, both the strength and durability of the concrete will be improved.

Table 5.—Laboratory freezing and thawing[1]

Silicone, tot G solids	Air	Durability factor [2]	Relative durability factor [3]
Percent by weight of cement	*Percent*	*Percent*	*Percent*
Control:			
None	5.1	83	------
A:			
0.05	5.8	92	111
B:			
0.01	5.3	91	110
.02	5.5	90	108
.05	5.7	93	112
.10	4.9	83	100
.30	5.0	82	99
.50	5.3	81	98
C:			
0.02	5.0	82	99
.05	5.2	80	96
.20	6.5	87	105
.50	6.1	74	107
E:			
0.01	5.9	91	110
.04	6.4	89	107
F:			
0.02	7.2	94	113
.10	6.8	92	111
.20	6.0	75	90
G:			
0.20	9±	81	98

[1] Each value is an average of tests on three 3- by 4- by 6-inch beams. Beams were frozen and thawed in accordance with ASTM Method C 291.

[2] Durability factor based on loss in N^2 after 1,000 cycles of freezing and thawing.

[3] Relative durability factor is the ratio in percent of the durability factor of the concrete containing silicone to the durability factor of the control concrete made on the same day.

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Relation of Asphalt Ductility To Pavement Performance

BY THE MATERIALS RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

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This article provides a critical review and restudy of published data as well as previously unpublished data concerning the relation of asphalt ductility to pavement performance.

The author points out that the consistency at which the asphalt begins to lose ductility rapidly and the temperature at which such consistency occurs is a significant relationship. The analysis of the data further indicates that, although an accelerated laboratory test is not available to accurately predict the ductility-penetration relationship of an asphalt in service, the ductility-penetration curve of the thin film residue provides a useful means for differentiating between asphalts and detecting those materials most likely to give unsatisfactory service. Requirements for minimum ductility, at 77° F., of the residue from the thin film oven test serve to eliminate potentially unsuitable materials.

Introduction

SINCE ITS introduction in 1903 the ductility test for asphalts has been, and still is, controversial. Some asphalt technologists believe that the test is an indication of a necessary property of asphalt related somewhat to its adhesive properties or stickiness, but others consider the present laboratory test for ductility of no value for indicating the potential quality of an asphalt as a paving material.

A review of the literature offers support for both of these divergent views. These contradictions suggest a need for a careful evaluation of the significance of the ductility test and its relation to other properties of the asphalt cement and a restudy of some of the available data to determine if there is a satisfactory explanation for the opposing viewpoints.

This article, which is part of a general symposium on the properties of asphalt that affect pavement performance, emphasizes the advantages of considering the ductility-penetration relationship of an asphalt in evaluating the effect of the asphalt characteristics on pavement performance. When available data are analyzed on this basis, there is a strong indication that the consistency at which the asphalt begins to lose ductility rapidly and the temperature at which such consistency occurs is a significant relationship. Because for some asphalts this point occurs at a relatively low penetration (or temperature) indications are that factors other than ductility, as measured in the laboratory test, control pavement performance, hence the conclusion is often reached that ductility is unimportant.

Summary

The amount of hardening of the asphalt during construction and the rate of hardening in service are the primary factors affecting durability of a pavement. However, the data discussed in this article demonstrate that the accompanying decrease in ductility of the asphalt is an important secondary factor that must not be overlooked. Pavements containing asphalts having penetrations in the range normally considered satisfactory (30 to 50) but having low ductilities are likely to show poorer service than pavements containing asphalts of the same penetration but having high ductilities.

The physical characteristics of the pavements, such as void content and permeability, and the environmental factors greatly affect the hardening rate of the asphaltic binder as well as the degree of oxidation during pavement service. Consequently, an accelerated laboratory test to accurately predict the ductility-penetration relationship or the change in the ductility-penetration relationship of an asphalt in service is not available. However, the ductility-penetration curve of thin film residue provides a useful means for differentiating between asphalts and detecting those materials most likely to develop unsatisfactory characteristics. Requirements for minimum ductility of the thin film residue at 77° F. based on the critical curve illustrated in this article serve to eliminate such potentially unsuitable materials. Recently adopted limits for ductility of the thin film residues in the AASHO specifications are based on this curve. These specifications have a minimum of 50 cm. for thin film residue of the 60–70 grade, 75 cm. for the 85–100 grade, and 100 cm. for all softer grades.

Further research is needed to establish the optimum conditions of the ductility test. The conditions of the 77° F. test at 5 cm. per minute may not provide the most useful information. Other temperatures, speeds of pull, or even shapes of specimens may prove to be more useful, but, until research data are available to define the optimum condition of the test, test requirements for the ductility of the thin film residue at 77° F. should be retained.

More research is also needed to clearly define the significance of ductility in relation to pavement behavior. An asphaltic pavement is subject to extremely wide temperature changes in service and the ductility of the asphalt, as measured by the present laboratory test, can vary from zero to values exceeding the limits of the ductilometer. Therefore, it is most likely that the ability of the asphalt to undergo elongation is not the primary characteristic affecting durability but rather that the ductility test result is an indication of an internal phase relationship of the asphaltic

[1] Presented at a meeting of the Association of Asphalt Technologists, San Francisco, Calif., February 1963.

constituents, which in turn have an important bearing on the serviceability factors of the asphalt.

Established Ductility-Penetration Relationships

The general relations of the ductility test results to test temperatures and consistencies of the asphalt and the variations of these relationships according to source or method of refining of the asphalt have been shown in published reports.

Figures 1 and 2, taken from a report by Lewis and Welborn presented before the Association of Asphalt Paving Technologists in January 1940 (*1*) [2] show respectively the variation of ductility in relation to changes in test temperatures and the penetrations of the asphalts. The significant feature of these curves is the many queer shapes that were obtained for asphalts from different sources when smooth curves were drawn to fit the data points. The differences in shape could be considered as indicative that the ductility as measured by the laboratory test at 77° F. has no true significance, but closer consideration shows that essentially all the curves have some common characteristics.

The curves for each asphalt manufactured by vacuum or steam reduction show that ductility increased sharply as test temperatures or penetrations were increased until a maximum was reached; upon further increase in temperature or penetration the ductility decreased, but usually at a slower rate. Ductility of some asphalts increased more gradually than others in relation to temperature. Although information about the manufacturing processes for these asphalts is incomplete, some oxidation is believed to have been used.

When the curves are viewed separately, no significant difference is apparent between the shapes of the curves for ductility against temperature shown in figure 1 or for ductility against penetration shown in figure 2. However, the relative positions of the curves for different asphalts are different and this is important when the behavior of one asphalt is being compared with another, particularly if the materials have significantly different consistencies at the same temperature.

Lewis and Welborn in the same article (*1*) illustrated another very important characteristic of asphalt ductility that is often overlooked. They showed a plot that included data from ductility tests made at six different temperatures on each of 10 asphalts of different grades that had been steam and vacuum reduced from the same crude source. The test temperatures ranged from 50° to 95° F. and the penetrations at 77° F. ranged from 23 to 182. The total range in penetration (extrapolated) was 6 to 442. The upper curve of figure 3 shows the test results for asphalts that had penetrations of less than 70. Despite the wide differences in ductility

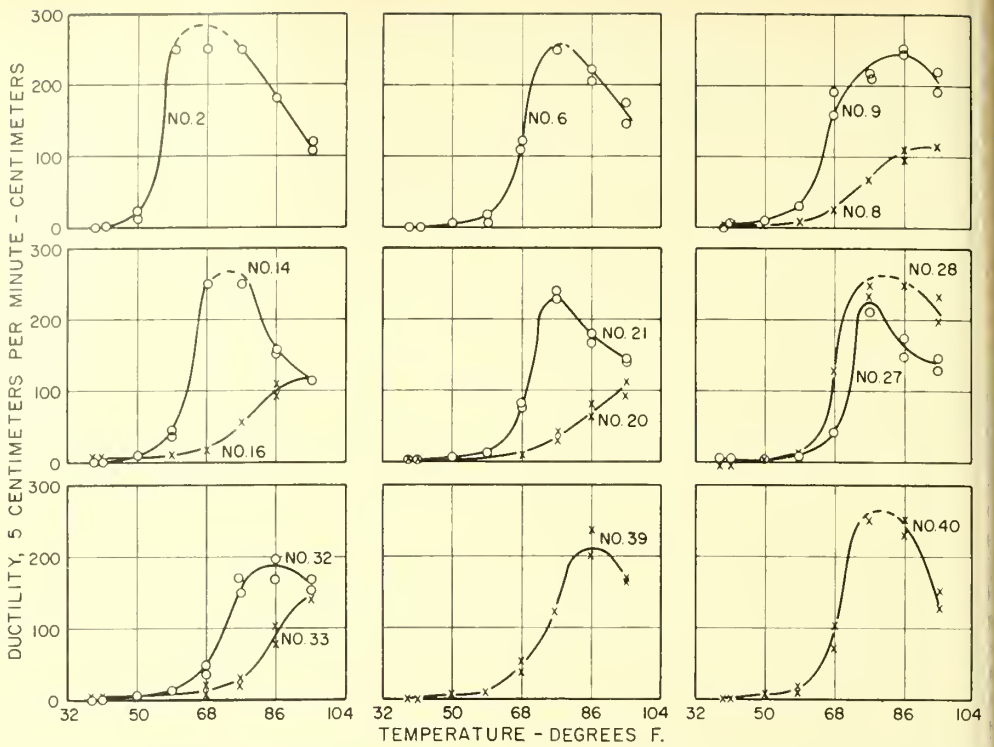

Figure 1.—Relation between ductility and test temperatures of selected samples of 50-60 penetration asphalts.

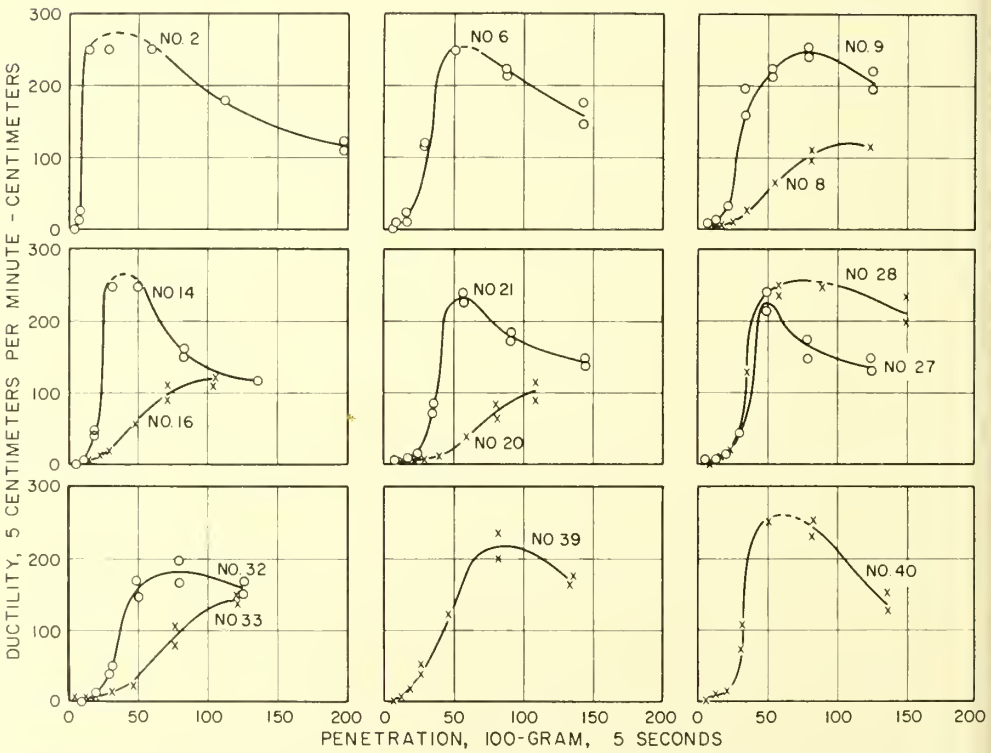

Figure 2.—Ductility-penetration relation of selected samples of 50-60 penetration grade asphalts, tested at different temperatures.

[2] References indicated by italic numbers in parentheses are listed on page 230.

at the different temperatures or for the different grades of asphalt at the same temperature, the data form a single ductility-penetration curve. The lower curve in figure 3, based on data reported by Lewis and Halstead (*2*), shows that the residues from the thin film oven tests for these same asphalts also form a single curve, even though the asphalts were of different grades and tests were made at different temperatures. This same relationship is generally true for all series of steam or vacuum reduced asphalts refined from the same source. However, when oxidation (blowing), cracking, or other refinery techniques are employed, Lewis and Halstead showed that the ductility for asphalts having the same penetration may differ significantly according to the grade of the asphalt. Figure 4 shows data for the ductility-penetration relationships for original asphalts and residues from the thin film test for each of three grades of asphalts obtained from the same source and that had positive spots in the Oliensis test. The data for each grade of material form a separate curve, but each of the curves for the residue is below the curve for the original asphalt of the corresponding grade.

Additional data to illustrate the basic relations between the ductility and penetration of asphalts after different treatments are available from unpublished test results obtained in 1940 by Committee 3A of the Association of Asphalt Paving Technologists (AAPT). In this cooperative effort, several laboratories, including the Bureau of Public Roads, conducted studies of nine asphalts. Test included the determination of the penetration and ductility of (1) the original materials, (2) the asphalts after several accelerated weathering tests, including oven heating and oxidation, and (3) the asphalts after their recovery from laboratory mixtures made and aged under different conditions. Figure 5 shows the data obtained by the Bureau of Public Roads for the ductility-penetration relationship of asphalts from two sources—a California crude and a midcontinent crude; both had been refined by the steam and vacuum process. The data points shown were obtained by use of a 50–60 and an 85–100 grade asphalt from each source.

Only a limited number of definite data points are available for the California asphalts as most of the ductility results were more than 250 cm. However, the available points illustrate the extremely rapid decrease in ductility known to be typical of asphalts from this source. Different conditions of hardening the asphalts did not materially affect the penetration at which the rapid decreases in ductility occurred.

The data points for the midcontinent asphalts show that the asphalts from all tests retained approximately the same ductility-penetration relationship. However, the best agreement with the general curve shown is indicated by the data for the 5-hour, thin film test for the asphalts recovered from the pavement samples and asphalts recovered from laboratory mixtures aged under different conditions. The significant deviations from the general curve can be explained by the degree of oxidation that occurred under the different conditions. Data points for the original asphalts and residues from the standard loss test, where oxidation was very limited, generally fall above the plotted curve. Data for the 18-hour thin film and oxidation residues, tests in which oxidation can be a major factor, fall below the plotted curve.

Interpretation of Ductility-Penetration Relationships

The relationships given in figures 1 through 5 provide a basis for the interpretation of ductility data from tests made on the original asphalts; on the residues from accelerated weathering tests, such as the thin film oven test; and on asphalts recovered from pavement samples. These figures illustrate that asphalts from the same source have the same ductility for equal consistencies, unless oxidation or other significant changes have occurred

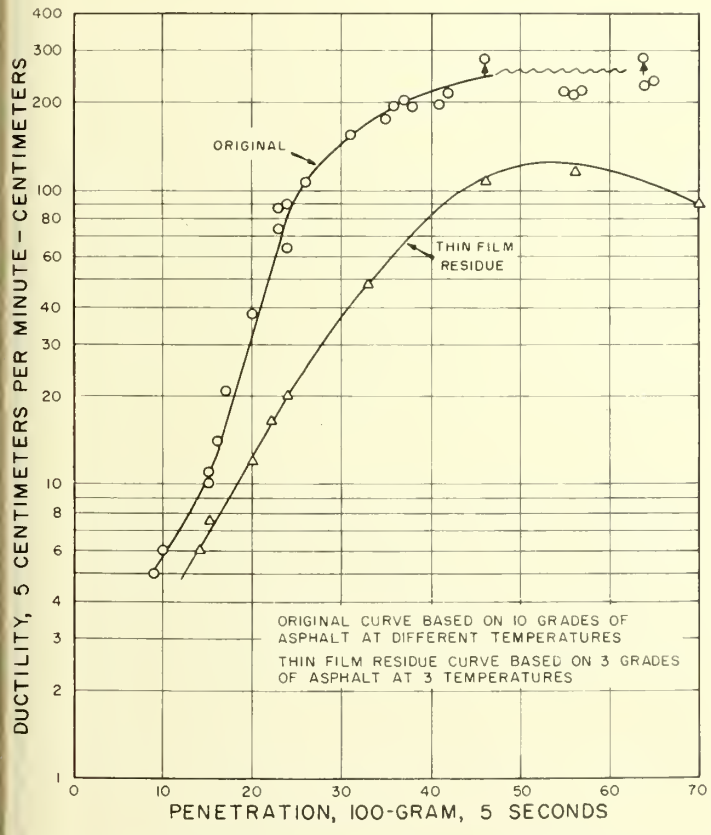
DUCTILITY, 5 CENTIMETERS PER MINUTE—CENTIMETERS
PENETRATION, 100-GRAM, 5 SECONDS
ORIGINAL
THIN FILM RESIDUE
ORIGINAL CURVE BASED ON 10 GRADES OF ASPHALT AT DIFFERENT TEMPERATURES
THIN FILM RESIDUE CURVE BASED ON 3 GRADES OF ASPHALT AT 3 TEMPERATURES

Figure 3.—Ductility-penetration relation for asphalts of different grades refined from the same crude petroleum by steam and vacuum distillation.

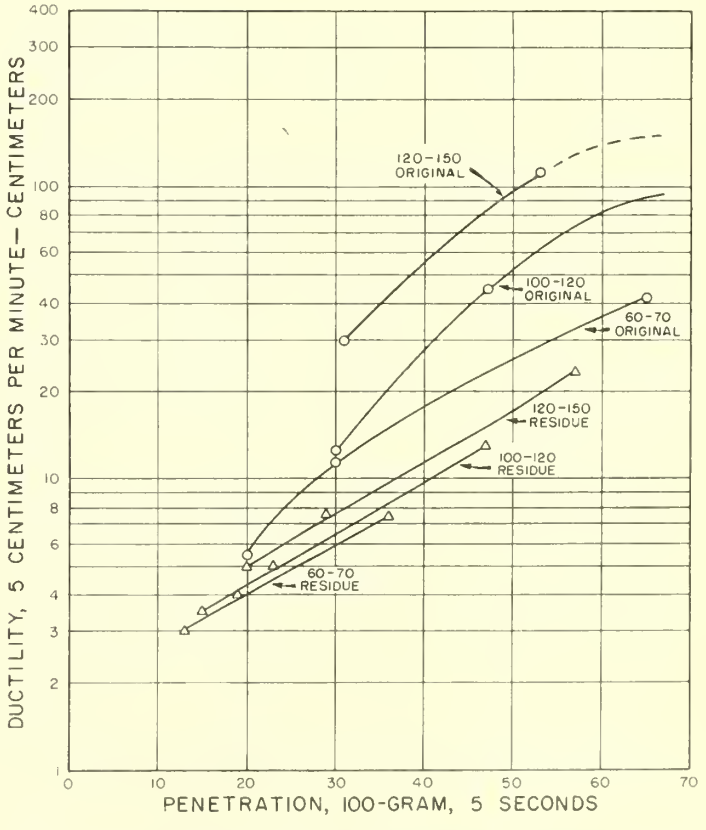

Figure 4.—Ductility-penetration relation for asphalts of different grades prepared from the same crude that had some cracking and blowing during the manufacture.

Table 1.—Characteristics of asphalts recovered from Kansas Experimental Projects

Sample	Age when sampled	Penetration at 77° F., 100 g., 5 sec.	Ductility, 5 cm. per min. at 77° F.	Performance [2] rating at 72 months
	Months		*Cm.*	
Asphalt *A*:				5
Composite [1]	10	62	200+	
Do	26	48	197	
Do	27	65	178	
Do	27	47	155	
Surface	72	35	127	
Binder	72	31	30	
Asphalt *B*:				5
Composite [1]	10	84	200+	
Do	26	69	200+	
Do	27	62	200+	
Surface	72	44	250+	
Binder	72	48	250+	
Asphalt *C*:				4
Composite [1]	10	64	200+	
Do	26	57	200+	
Do	27	65	200+	
Surface	72	48	250+	
Binder	72	52	250+	
Asphalt *D*:				9
Composite [1]	10	46	10	
Do	26	40	8	
Do	27	40	9	
Surface	72	28	4	
Binder	72	29	4	
Asphalt *E*:				5
Composite [1]	10	98	168	
Do	26	59	150	
Do	27	65	178	
Surface	72	43	71	
Binder	72	48	94	
Asphalt *H*:				7
Composite [1]	10	63	194	
Do	26	61	154	
Do	27	43	62	
Surface	72	34	14	
Binder	72	34	18	

[1] Asphalt from both surface and binder course recovered in same sample. Data obtained by Kansas State Highway Commission.
[2] A rating of 1 is excellent; a rating of 10 denotes complete failure.

Table 2.—Characteristics of asphalts recovered from Virginia Experimental Project

Sample	Test temperature	Pavement age	Penetration, 100 g., 5 sec.	Ductility, 5 cm. per min.
Asphalt *A*:	° *F.*	*Months*		*Cm.*
Original	77		80	250+
Do	60		27	250+
Do	50		13	191
Do	45		9	46
Thin film residue	77		47	250+
Do	60		13	89
Do	50		9	8
From pavement [1]	77	0	50	250+
Do	77	12	33	250+
Do	77	24	34	250+
Do	77	48	28	250+
Asphalt *B*:				
Original	77		71	250+
Do	60		28	154
Do	50		15	16
Thin film residue	77		44	170
Do	60		18	17
Do	50		11	5
From pavement [1]	77	0	48	242+
Do	77	12	38	125
Do	77	24	33	64
Do	77	48	32	50
Asphalt *C*:				
Original	77		82	239
Do	60		31	229
Do	50		19	35
Thin film residue	77		57	195
Do	60		21	26
Do	55		15	9
Do	50		11	6
From pavement [1]	77	0	64	238+
Do	77	12	45	165
Do	77	24	39	135
Do	77	48	40	128

[1] Figures given are averages of results from four samples taken laterally across the pavement from each section.

in the asphalt composition. Thus, th difference between ductility at the same cor sistency for original asphalts and ductility c asphalts after accelerated tests or aft recovery from the pavement is a measure c the ductility destroyed or lost. This ductilit decrease can also be considered a measure c the degree of change caused by oxidation c other alterations in composition of asphalt The ductilities for original asphalts at equiva lent penetrations can be obtained by makin tests at several temperatures to establish th ductility-penetration curve.

Because of the conditions of the thin fil oven test, the change in the ductility-penetra tion relation occurring during the test woul be expected to be equal to or less than chang in ductility-penetration occurring in the pav ment in use. The loss of ductility during th thin film test may then be considered t represent the minimum change expected i service and asphalts that failed to retai adequate ductility in this test would mos likely undergo rapid decreases in ductility i the pavement. For asphalts retaining hig ductility at relatively low penetration aft the thin film test (for example, ductility c more than 150 cm. at penetrations as low a 20) loss of ductility measured by the labora tory test most likely can be discounted as factor in the pavement behavior. Howeve cracking and ravelling caused by abnorma hardness (low penetration) for such materia may still be a contributing factor in pavemen performance.

Critical Ductility-Penetration Relationship

The foregoing discussion implies that critical ductility-penetration relationshi exists below which low ductility would be potential cause of poor pavement service an above which ductility would not be a signif cant factor affecting pavement durabilit Admittedly, because of the many variable involved, a precise location for such a dividin line will be extremely difficult to determin However, examination of the ductility-pen tration relationships of asphalts used in man pavement evaluation studies, including m terials from many of the more importa sources of highway asphalts, indicates th most asphalts will have ductilities for equiva lent penetrations that will plot above th dotted curve shown in figures 6 through 1 As will be discussed later, considerab evidence indicates that a ductility-penetratio relationship that plots in the area below th curve may have a bearing on performance an asphalt in service. The limiting valu of ductility for the thin film residues include in the recently adopted AASHO specificatio for asphalt generally are based on relatio ships shown by the dotted line in figs. 6–1

Relation of Pavement Performanc to Penetration-Ductility Relationship

To determine the extent to which th penetration-ductility relationship is a fact in pavement behavior, both published an

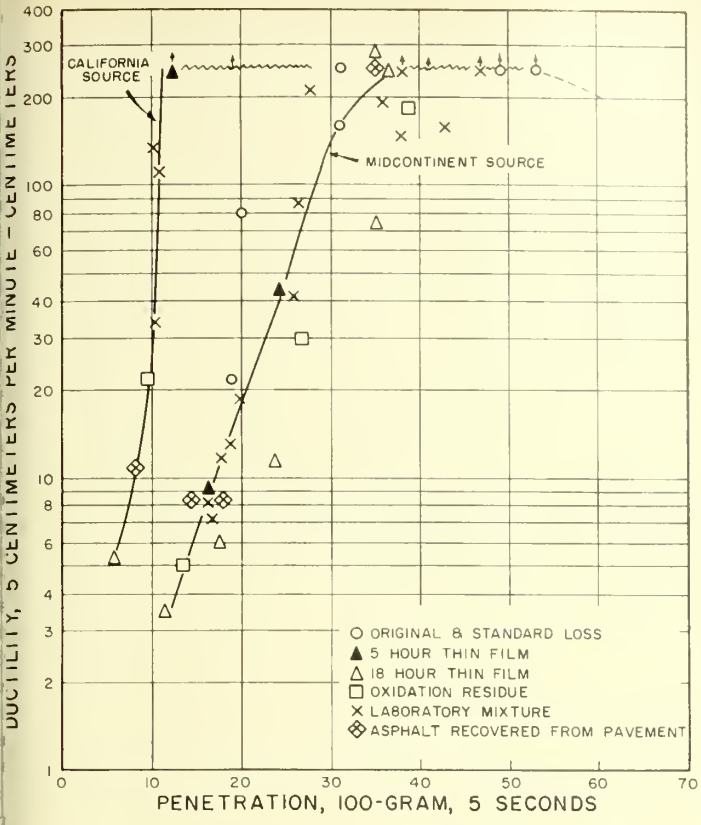

Figure 5.—Ductility-penetration relation of residues of steam and vacuum refined asphalts from the same source after hardening under different conditions. Tests were made at 77° F.

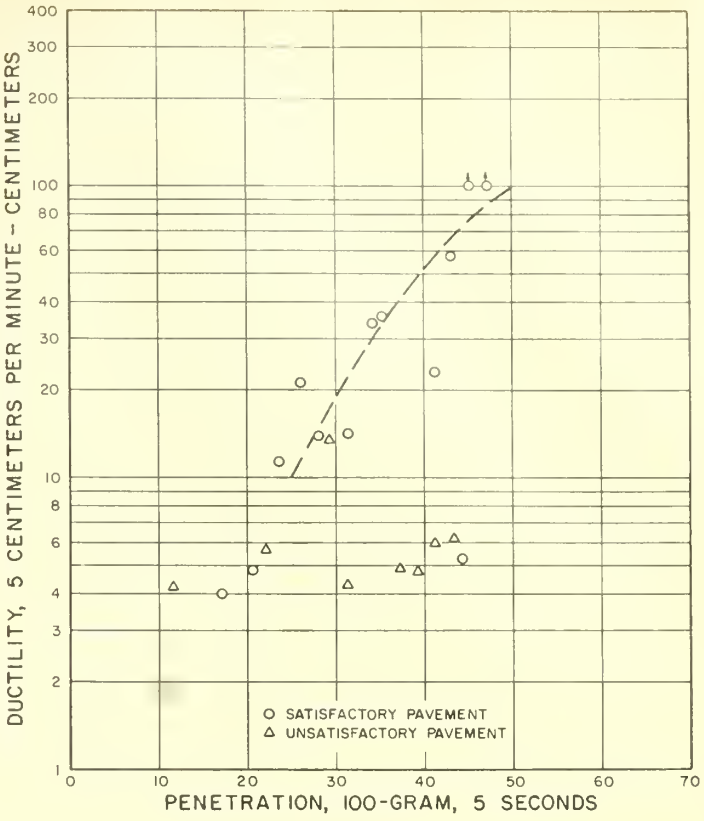

Figure 6.—Ductility-penetration relation of asphalts recovered from Ohio pavements. Tests were made at 77° F.

unpublished data from several pavement projects were examined.

Ohio study

In 1941 the Bureau of Public Roads and the Ohio Department of Highways made a joint study of bituminous concrete pavements in Ohio (*3*) in which factors affecting pavement performance were considered. One of the conclusions was that, for the 50–60 penetration asphalts used in the asphaltic concretes, satisfactory service was not likely to continue after the penetration of the asphalt had fallen appreciably below 30. The corresponding critical ductility was reported to be about 10 to 13 centimeters. The report on the Ohio pavements contains ductility and penetration data at 77° F., plotted in figure 6. As can be seen from this figure, unsatisfactory service in Ohio was generally associated with a low ductility at penetrations between 30 and 50. The only significant departure from this trend being one project that was reported to be in good condition after 35 months of service, even though the ductility was 5 centimeters for a penetration of 44. It is coincidental that the dotted line indicating the location of the critical ductility-penetration relationship appears to be the locus of the data points for the satisfactory pavement. However, the important relationship indicated is that, although several satisfactory pavements fall below the ductility-penetration boundary, no unsatisfactory pavements having penetrations greater than 25 were above the boundary.

Kansas study

Data given table 1 and plotted in figure 7 illustrate previously unpublished test results obtained from an experimental project in Kansas. This project involved six asphalts from different sources that were used as an overlay over old rigid pavement. The asphalts were recovered from samples cut periodically from the pavement. The curves for each asphalt were obtained by plotting the ductility at 77° F. against the penetration at 77° F. of the asphalts recovered from the pavement at different ages. The relative 6-year service rating of the sections containing each asphalt are shown in table 1. In the rating system used, a rating of 1 indicates no failure or cracking whereas a rating of 10 indicates complete failure.

The section containing asphalt *D*, for which the ductility was very low at penetrations in the range of 30–40, had the poorest service record. The section containing asphalt *H* also had low ductility after the penetration had dropped below 30, and it showed the next poorest service. Asphalts *A*, *B*, and *C* all retained ductilities of more than 100 cm. as well as relatively high penetrations, and the service records for pavements containing these asphalts were generally satisfactory. The penetration-ductility curve for asphalt *E* was approximately the same as that for asphalt *H* although the service records were significantly different. However, as indicated, asphalt *E* did not harden as much as *H* in service, thus it did not fall below the critical ductility-penetration relationship and the pavement containing it performed satisfactorily.

The data points for the ductility-penetration relationship of the thin film residues, shown as solid dots on figure 7, fall close to the curve for the asphalts recovered from the pavement except for those for asphalt *D*. The thin film data points for this asphalt fall significantly below the curve, and this low ductility indication of poor service was verified by actual performance of the pavement. Asphalt *H* would have been considered satisfactory on the basis of present criteria of ductility of the thin film residue at 77° F. but as it was a borderline material on the basis of the ductility-penetration boundary suggested in this article, its relative rapid rate of hardening apparently contributed to poor pavement service.

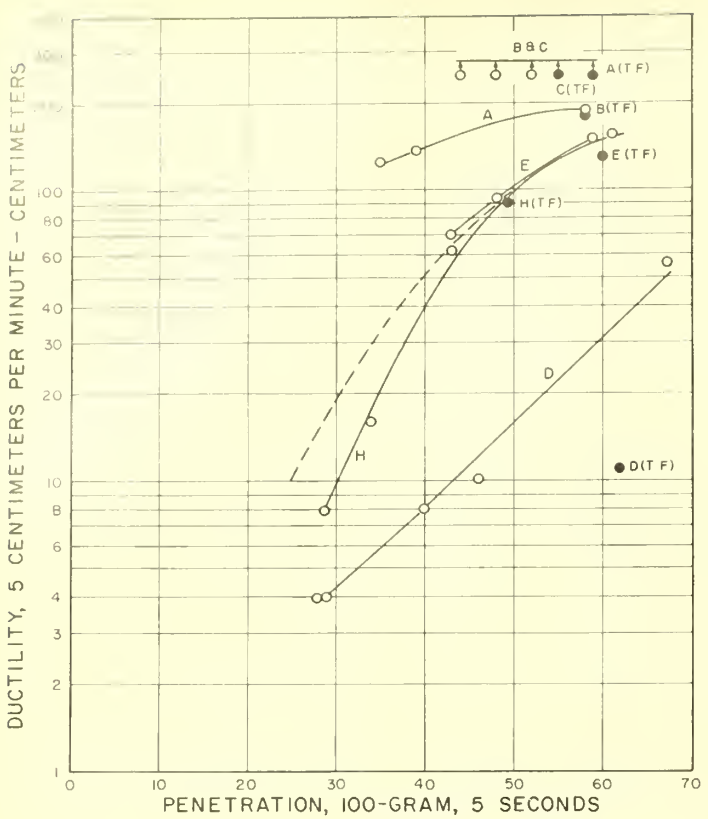

Figure 7.—Ductility-penetration relation of asphalts recovered from Kansas pavements at different ages. Tests were made at 77° F.

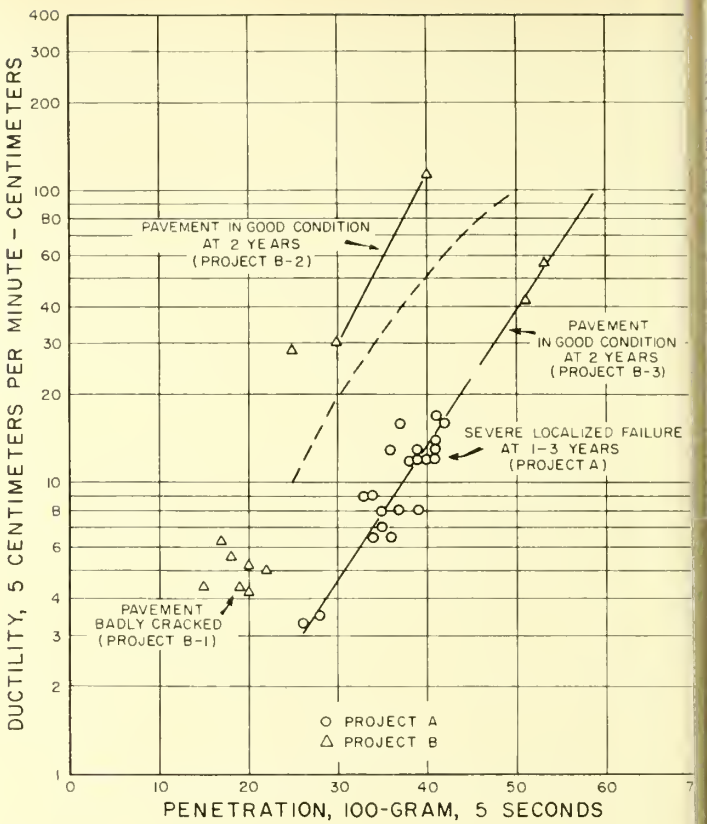
DUCTILITY, 5 CENTIMETERS PER MINUTE – CENTIMETERS
PENETRATION, 100-GRAM, 5 SECONDS
PAVEMENT IN GOOD CONDITION AT 2 YEARS (PROJECT B-2)
PAVEMENT IN GOOD CONDITION AT 2 YEARS (PROJECT B-3)
SEVERE LOCALIZED FAILURE AT 1-3 YEARS (PROJECT A)
PAVEMENT BADLY CRACKED (PROJECT B-1)
PROJECT A
PROJECT B

Figure 8.—Ductility-penetration relation of asphalts recovered from pavement projects. Tests were made at 77° F.

Two Public Roads studies

Figure 8 illustrates data obtained by Public Roads from two extensive studies of pavements. Project A, shown by the lower curve, was subject to considerable criticism because of limited areas of severe failure that occurred between 1 and 3 years of service. The study of this project indicated a relatively low asphalt content in relation to the amount of fine aggregate present, and some localized base failures were also observed. However, the low ductility of the asphalts at relatively high penetrations is believed to have contributed significantly to the rapid failure of the pavement observed in localized areas once initial cracking occurred.

Project B indicated by the triangular data points in figure 8 was a low-traffic road built over a period of years by several contractors. The older sections of this project designated on the curve as project B–1 were badly cracked although riding qualities remained satisfactory. Each data point shown is the average of the results of several samples from each section, which represents a construction contract. For the badly cracked areas, the penetrations at 77° F. ranged from 15 to 22 and the ductility at 77° F. ranged from 4 to 6 cm. Extreme hardness of the asphalt in these sections undoubtedly was the main factor contributing to poor performance. Overheating of the asphalt during construction or the effects of relatively high voids and very little traffic could have been the cause of this hardening. The data points obtained from two newer sections of project B that are still in good condition are of interest. These sections were built with different asphalts. Although the asphalt in project B–2 has generally hardened more than that in project B–3 the ductility-penetration relation for the asphalt in project B–2 is superior to that of the asphalt in project B–3. Consequently, if the asphalt hardening in project B–3 continues, this project may fail earlier than project B–2.

Bissett report

The data plotted in figure 9, taken from the report presented at the 1962 Highway Research Board meeting by Professor J. R. Bissett (*4*) illustrates one of the advantages of considering the ductility-penetration relationship in analyzing data obtained from pavement samples.

Professor Bissett's report included data from five projects using Brand *C* asphalt. These pavements were of different ages and differed in service performance. When the data were examined independently, greater differences in ductility for the different recovered asphalts than would be accounted for by differences in penetrations appeared t exist but, as shown in figure 9, when ductilit is plotted against penetration most of th points fall close to the same curve. The dat plotted in figure 9 are for four of the fiv projects for which Brand *C* asphalt was use and for the one project in which Brand asphalt was used, except that several duplic tions of data points at the same location a not indicated. The curves plotted in figu 9 indicate that differences in behavior pavements of these projects were not cause by any fundamental differences in the asphalt Variations in construction that permitted mo asphalt hardening in some instances than others were most likely the cause. Also, a of the data for Brand *C* asphalt show relativel low ductilities for equivalent penetratio compared to asphalt from other sources; thu accelerated pavement deterioration ma occur in all of these projects once failure begins.

The data for the project in which Brand asphalt was used indicate wide variations i ductility for essentially the same penetratio but, as illustrated in figure 9, the data point are in the area where many asphalts show very rapid decrease in ductility for sma decreases in penetration. Therefore, the di ferences were most likely the normal result greater hardening and experimental erro rather than of a significant loss in ductilit for equivalent penetration.

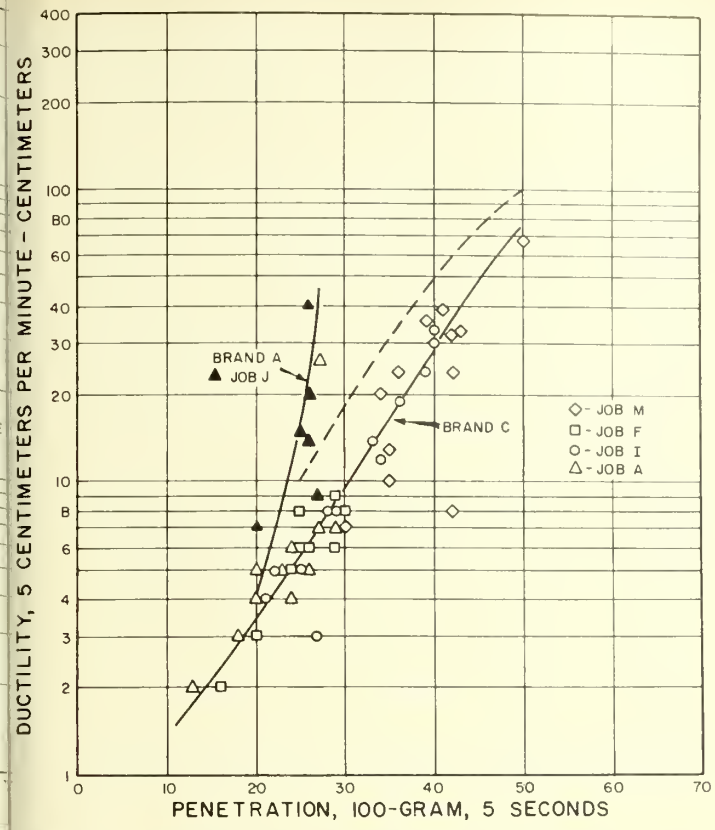

Figure 9.—Ductility-penetration relation of asphalts recovered from pavement projects in Arkansas. Tests were made at 77° F.

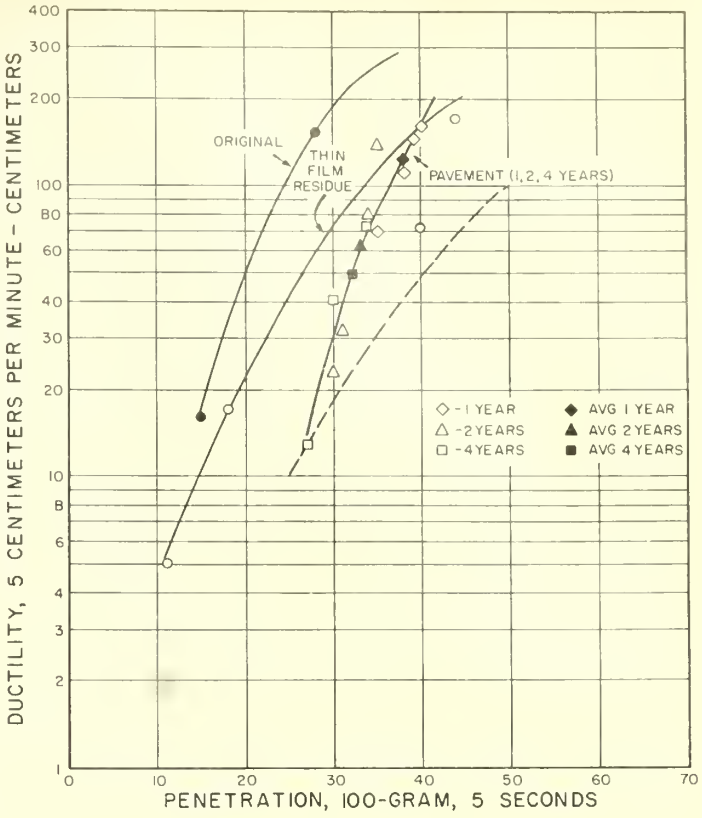

Figure 10.—Ductility-penetration relation for asphalt B used in Virginia experimental project. Tests on original asphalts and thin film residues were made at different temperatures, and those on asphalts recovered from pavement were made at 77° F.

The penetration-ductility relationship for the asphalts recovered from the pavement projects discussed in the preceding paragraphs are based on tests made at 77° F. only. However, the implications from data shown in figure 5 are that ductility and penetration tests made at several different temperatures for the same original material and for the thin film residue would provide data for ductility-penetration curves that could possibly be used as the basis for predictions of pavement service changes. To indicate the usefulness of this approach, the ductilities and penetrations of the asphalts being used in experimental projects now under study were determined at several different temperatures.

Table 2 shows ductility and penetration data for the original asphalts, the thin film residues, and asphalts recovered from the pavement at different ages for materials used in an experimental project in Virginia; and table 3 shows similar data for 10 asphalts used in the Zaca-Wigmore California Project. The significant ductility-penetration relationships indicated by these data are shown in figures 10–15.

Virginia study

Because asphalt *A* of the Virginia Project retains ductility in excess of 250 cm. after 4 years in service, trends for the relation of penetration and ductility are not illustrated. However, the data for asphalts *B* and *C* shown in figures 10 and 11 illustrate that significant changes are occurring in these asphalts. As would be expected on the basis of the principle discussed earlier in this report, the plotted curves show the ductility of the original asphalt to be greater than the ductility of the thin film residue for equivalent penetration. Although only the averages of test results for four samples—taken laterally across the road—are shown in table 2 for each pavement age, the individual results, as well as the averages, are plotted in figures 10 and 11. On the basis of the averages reported in table 2, no significant change occurred in the penetration of the recovered asphalt between 2 and 4 years, but the ductility decreased to some extent. The individual results in figures 10 and 11, however, are more informative. For example, in figure 10, all of the data points except the result for one 4-year sample are close to the plotted curve, but the points are not in sequence with respect to pavement age. This variation in rate of hardening in the same section was mostly likely caused by the differential effect of traffic at the edge of the road, in the wheel path, and between wheel paths. To date, there has been no significant failure in the pavement sections. Figure 10, however, indicates that the section containing asphalt *B* may be approaching a critical point in some areas, and further hardening and reduction of asphalt ductility may induce failure under severe weather or traffic conditions. The data for asphalt *C* plotted in figure 11 follows essentially the same trend as does the data for asphalt *A* in figure 10 except that the hardening and reduction of ductility have not progressed as far.

Zaca-Wigmore Project

The Zaca-Wigmore experimental project, constructed in 1954 and 1955 in California is one of the better known projects now under study and several reports have been issued concerning the performance of the asphalts (*5, 6, 7*).

The asphalts used in this project were of the 200–300 penetration grade, thus at 77° F. the ductilities of all the materials were generally high and very little useful information is provided by the results of the ductility tests made at 77° F. on the original asphalts. However, an analysis of test data obtained by the Bureau of Public Roads for ductilities of the asphalts and the thin film residues at different temperatures, together with data for the ductility and penetration of asphalts recovered from the pavements at different ages reported in 1959 by Hveem, Zube, and Skog (*5*), provides some interesting clues to

Table 3.—Characteristics of asphalts recovered from Zaca-Wigmore Project [1]

Sample	Test temperature	Pavement age	Penetration, 100 g., 5 sec.	Ductility, 5 cm. per min.
Section A:	° F.	Months		Cm.
Original	60		64	244
Do	50		27	250+
Do	40		14	250+
Thin film residue	77		120	193
Do	60		35	250+
Do	50		17	250+
Do	45		12	250+
Do	40		8	38
From pavement	77	5	131	
Do	77	12.5	84	
Do	77	20	49	
Do	77	35	44	100+
Section B-1:				
Original	60		82	180
Do	50		44	148
Do	45		30	174
Do	40		21	190
Thin film residue	77		121	152
Do	60		42	131
Do	50		24	87
Do	45		17	26
From pavement	77	3.5	148	
Do	77	11	107	
Do	77	19	77	
Do	77	33	56	100+
Section C:				
Original	60		66	250+
Do	50		32	175
Do	40		15	0
Thin film residue	77		120	191
Do	60		36	250+
Do	50		18	250+
Do	40		9	37
From pavement	77	5	132	
Do	77	12.5	79	
Do	77	20	46	
Do	77	35	44	100+
Section D:				
Original	60		74	221
Do	50		34	220
Do	40		14	250+
Thin film residue	77		121	173
Do	60		36	250+
Do	50		18	250+
Do	45		12	250+
Do	40		9	0
From pavement	77	5	151	
Do	77	12.5	93	
Do	77	20	59	
Do	77	35	52	100+
Section E:				
Original	60		85	238
Do	50		49	170
Do	45		35	243
Do	40		26	250+
Thin film residue	77		65	201
Do	60		25	138
Do	50		15	21
Do	45		11	9
From pavement	77	5	93	100+
Do	77	12.5	38	100+
Do	77	20	20	
Do	77	35	16	16
Section F:	° F.	Months		Cm.
Original	60		68	216
Do	50		35	174
Do	40		20	171
Thin film residue	77		85	224
Do	60		32	122
Do	50		16	22
From pavement	77	5	93	100+
Do	77	12.5	57	
Do	77	20	42	100+
Do	77	35	38	21
Section G:				
Original	60		80	186
Do	55		60	209
Do	50		44	103
Do	45		32	137
Do	40		26	60
Thin film residue	77		90	141
Do	60		36	41
Do	50		24	11
Do	45		18	7
From pavement	77	5	86	82
Do	77	12.5	47	
Do	77	20	39	7
Do	77	35	33	12
Section G-2:				
Original	60		74	242
Do	50		40	225
Do	40		21	200+
Thin film residue	77		75	183
Do	60		30	96
Do	50		17	16
Do	40		10	6
From pavement	77	7.5	62	100+
Do	77	16	52	100+
Do	77	30.5	34	96
Section H:				
Original	60		69	246
Do	50		33	198
Do	40		16	250+
Thin film residue	77		107	142
Do	60		36	250+
Do	50		16	250+
Do	45		12	109
From pavement	77	5	109	
Do	77	12.5	61	
Do	77	20	40	31
Do	77	35		100+
Section J:				
Original	60		77	242
Do	55		54	250+
Do	50		40	250+
Do	45		28	250+
Do	40		19	119
Thin film residue	77		127	184
Do	60		44	250+
Do	50		26	89
Do	45		19	21
From pavement	77	5	154	
Do	77	12.5	95	
Do	77	20	58	
Do	77	35	70	100+

[1] Data for original asphalts and thin film residues obtained by Bureau of Public Roads Tests. Data from pavement samples taken from report by Hveem, Zube, and Skog (*5*).

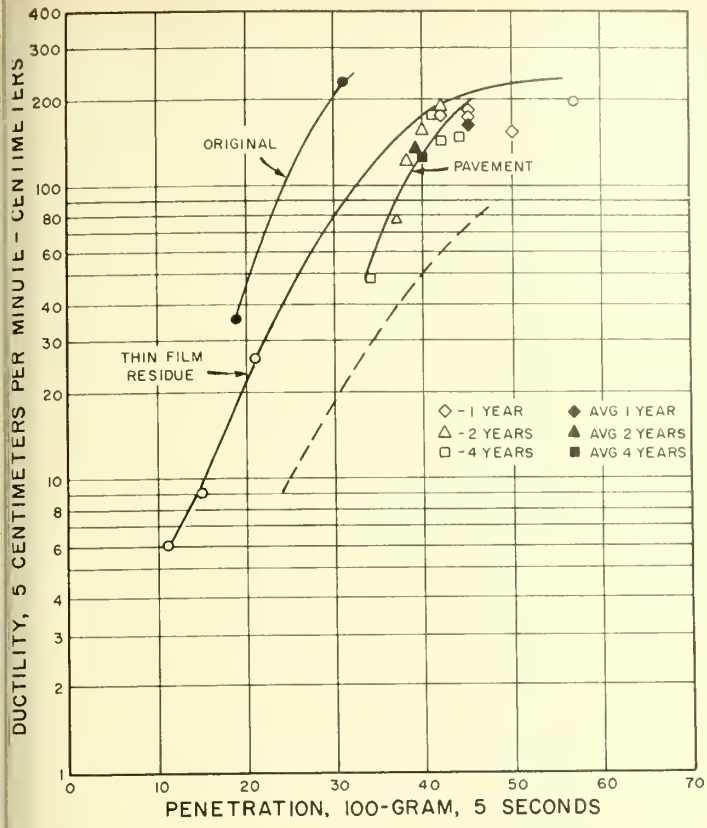

Figure 11.—Ductility-penetration relation for asphalt C used in Virginia experimental project. Tests on original asphalts and thin film residues were made at different temperatures, and those on asphalts recovered from pavement were made at 77° F.

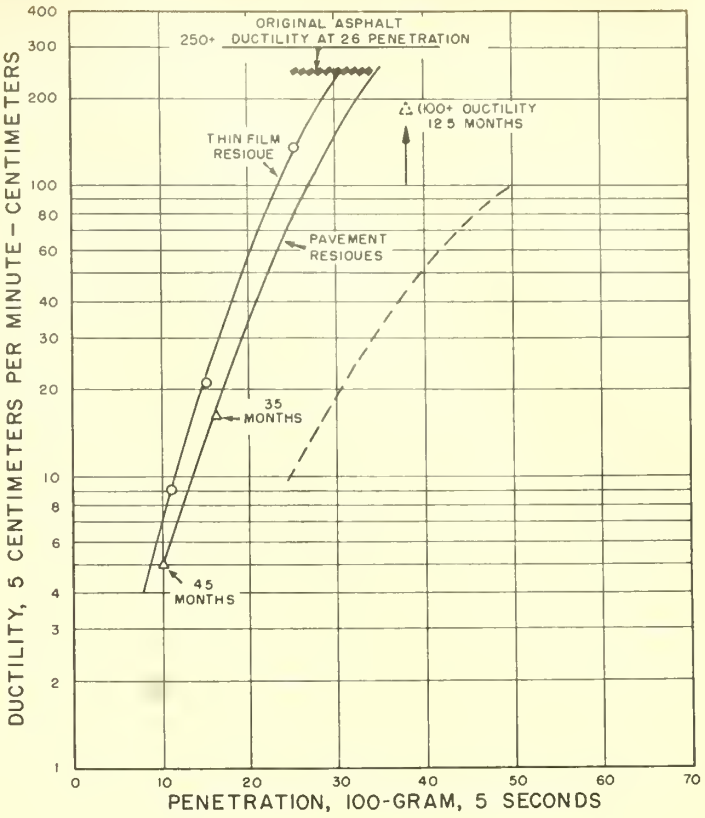

Figure 12.—Ductility-penetration relation for asphalt E in Zaca-Wigmore experimental project. Tests on original asphalts and thin film residues were made at different temperatures, and those on asphalts recovered from pavement were made at 77° F.

the asphalt characteristics. Table 3 shows the test data from both sources. Sections E, F, and G all showed relatively poor performance. Figure 12 shows the ductility-penetration relationships of the asphalts used in section E. This asphalt lost 4.45 percent by weight and retained only 27.7 percent of its original penetration in the thin film test, thus it would not comply with the asphalt specification now being used in California. The 1959 progress report showed that section E had failed in 1958. Figure 12 confirms the conclusion that the early failure of this section can be attributed almost entirely to the hardening caused by the high degree of volatility of this asphalt. The asphalts recovered from the pavement at 35 and 45 months had penetrations at 77° F. of 16 and 10 respectively but showed essentially the same ductility-penetration relationship as the thin film residue tested at different temperatures. It can be concluded from these test results that oxidation has not caused large changes.

The data reported for the asphalt used in section F and shown in figure 13 are more erratic. This asphalt also had a high loss (2.25 percent) and a low retained penetration (34.7 percent of the original) in the thin film test. The characteristics of the asphalts recovered from pavement samples at different periods suggest that this asphalt may be subject to rapid oxidation in the road as well as have considerable volatility. Based on the arbitrary dividing line, the asphalt recovered at 38 months had a relatively unsatisfactory ductility-penetration relationship. At 54 months the range of ductilities of the recovered asphalts was 7 to 83 cm. for 9 cores. The average penetration was 25. It is possible that the variation in penetration of the asphalt recovered from each core would account for a large proportion of the difference in ductility, but it is also possible that localized variations in conditions after construction permitted a variable amount of oxidation of the asphalt.

The ductility-penetration relationships shown in figure 14 for the asphalts used in sections G and G–2 are of interest. Hveem and his coauthors (*5*) reported that a change in the refinery methods or crude sources was made by the manufacturer of the asphalts used in these sections, but the reported differences in laboratory tests were small. However, the hardening that occurred during mixing for asphalt used in section G was considerably greater than for asphalt used in section G–2, the percentages of retained penetrations were 45.1 and 71.8, respectively.

The results of tests on the original asphalts and the thin film residue for asphalt from section G shown in figure 14 indicate that this asphalt is subject to considerable change in the ductility-penetration relationship during the thin film test and thus is also likely to undergo rapid changes in pavement service. The data for the asphalts recovered from the pavement confirm that changes in the ductility-penetration relationship had occurred. All of the ductility-penetration data points fall somewhat below the thin film residue line and also below the arbitrarily established critical boundary.

Although test results reported for the asphalt used in section G–2 indicated that it was similar to the asphalt used in section G, the test results plotted in figure 14 show that the thin film residue of the asphalt in section G–2 had a ductility-penetration relationship that was definitely superior to that of the thin film residue of the asphalt used in section G. The data point for the asphalt recovered from the pavement at 30 months shows that at this age the asphalt still retained a good ductility-penetration relationship. It is believed that a significant factor in the better performance of pavement in section G–2 reported in the 1959 progress report is this better ductility-penetration relationship as well as a slower rate of hardening of the asphalt.

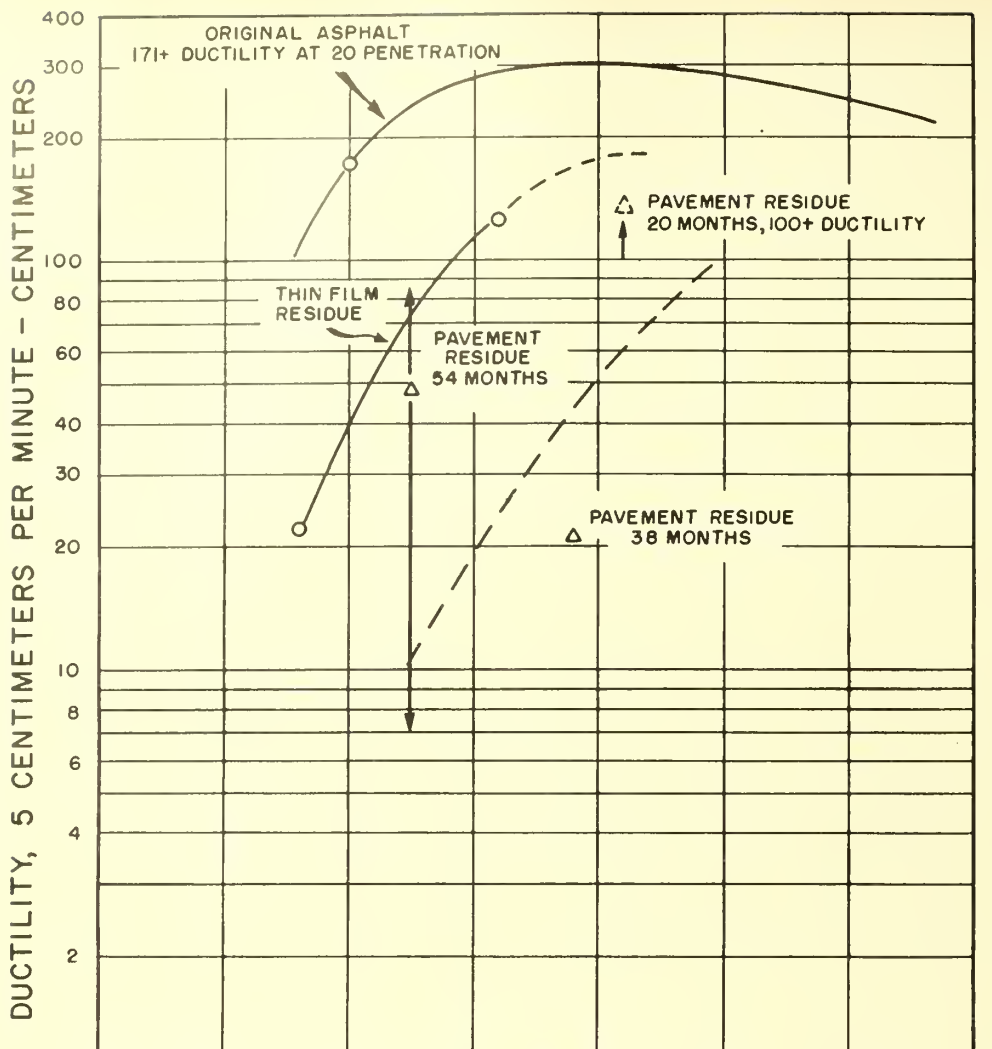

Figure 13.—Ductility-penetration relation of asphalt used in section F of Zaca-Wigmore experimental project. Tests on original asphalt and thin film residue were made at different temperatures, and those on asphalts recovered from pavement were made at 77° F.

Effect of difference in asphalt content

Two sections of the Zaca-Wigmore road were designated I–2. One of these contained 5.8 percent and the other 6.3 percent asphalt. The asphalts recovered after several periods of service were significantly different. The penetrations reported in 1959 for section I–2 containing 6.3 percent asphalt were 71, 55, and 51 when recovered at 7.5, 16, and 30.5 months respectively. Ductility, reported only for the 30.5-month sample, was 96 cm. Penetrations for the asphalts recovered from the section containing 5.8 percent asphalt were 56, 38, and 25 and had corresponding ductilities of 95, 10, and 6 cm. Although data for the thin film residues and original asphalts at temperatures other than 77° F. are not available for these sections, figure 15 shows that the ductility-penetration relationship for the sample at 30.5 months from the section containing 6.3 percent asphalt was essentially the same as the relationship for the sample obtained at 7.5 months from the section containing 5.8 percent asphalt. This indicates that the lower rate of hardening in the section containing the higher asphalt content was the result of increased protection against oxidation.

The asphalt used in section J was included in the California experiment because it was considered to represent a high quality product. The service record of this section was reported to be excellent in the 1959 progress report. The ductility-penetration relationships of this asphalt, based on tests at different temperatures and shown in figure 15, show that the original asphalt retained a high ductility for a relatively low penetration. All test results for ductility were more than 200 cm., except for the test made at 40° F. for which the ductility was 119 cm. and the penetration was 19. The ductility at 19 penetration for the thin film residue was 21 cm., considerably less than for the original but still well within the area considered satisfactory. The available data from asphalts recovered from the pavement section shows that at 35 months the ductility at 77° F. was greater than 100 cm. and the penetration was 70. Thus, the ductility and penetration of this asphalt in the pavement is not likely to become critical even at relatively low service temperatures.

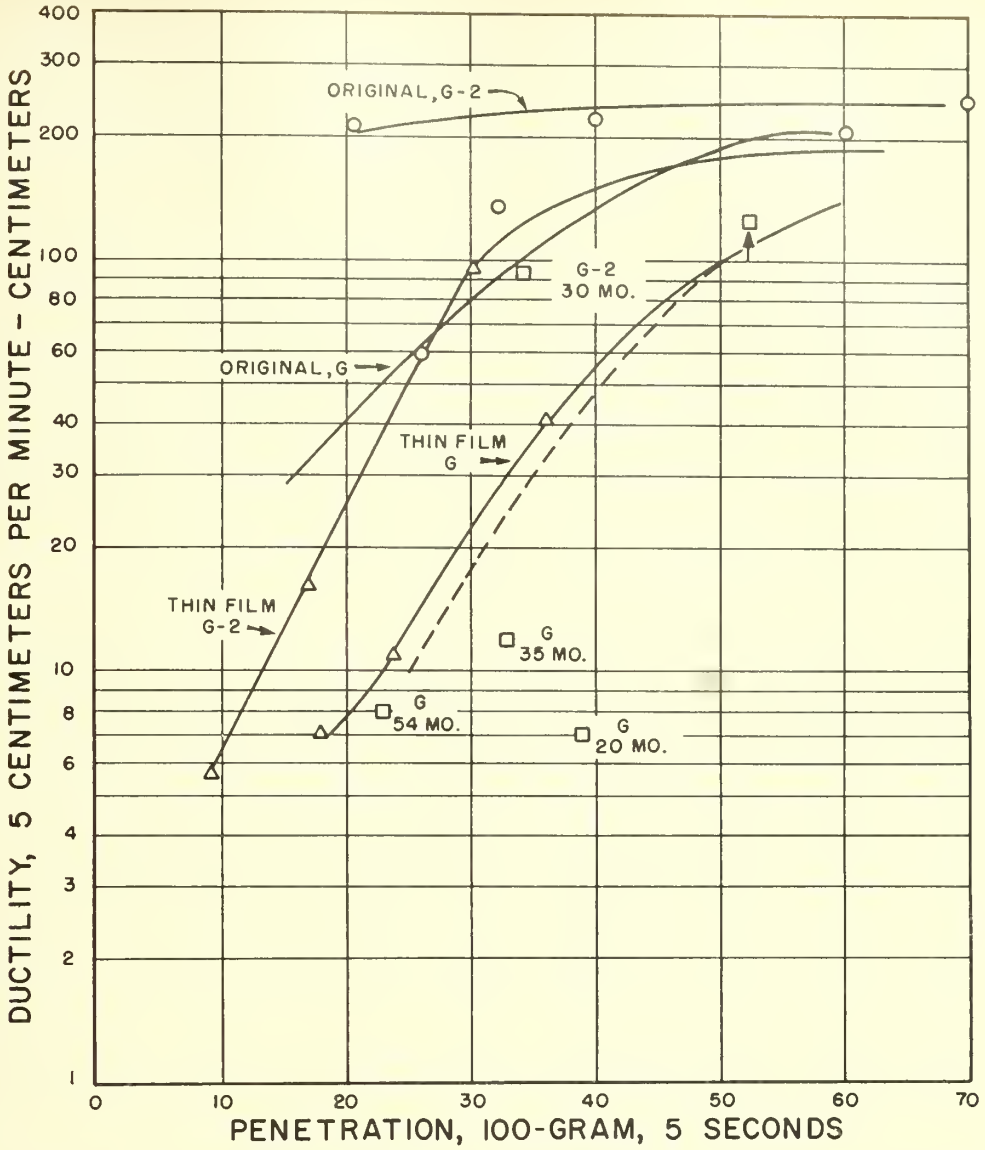

Figure 14.—Ductility-penetration relation of asphalts used in sections G and G-2 of Zaca-Wigmore experimental project. Tests on original asphalts and thin film residues were made at different temperatures, and those on asphalts recovered from pavement were made at 77° F.

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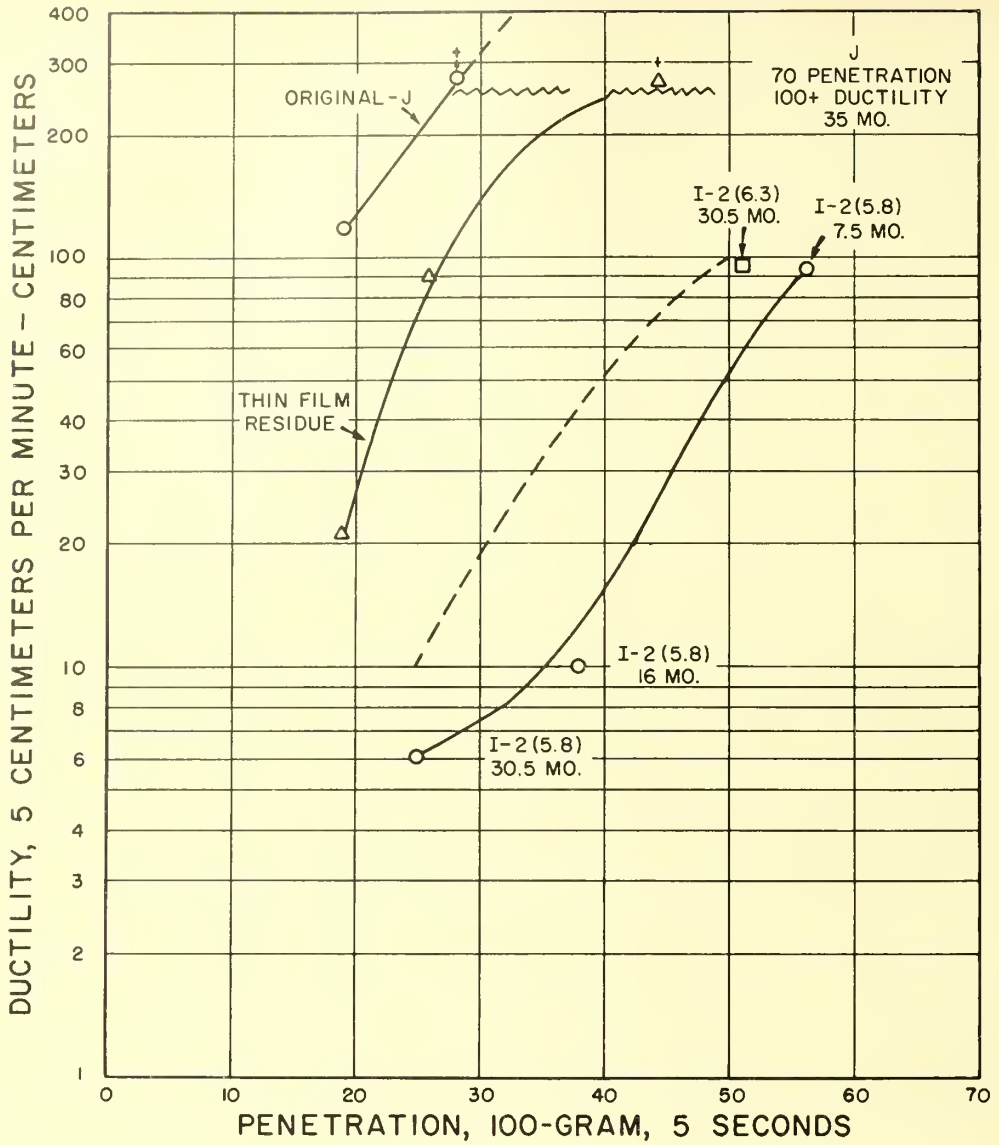

Figure 15.—Ductility-penetration relation of asphalts used in sections J and I-2 of Zaca-Wigmore experimental project. Tests on original asphalts and thin film residues were made at different temperatures, and those on asphalts recovered from pavement were made at 77° F.

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DECEMBER 1963

Public Roads

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Truck traffic on U.S. Route 10 at Lookout Pass on the Idaho-Montana State line, 62 miles east of Coeur d'Alene, Idaho. Final location of Interstate Route 90 over this pass is under study by the State highway departments. Present alinement to the summit is winding and has sharp curves. The maximum curvature is 28 degrees and the maximum grade is 7 percent; for long stretches the grade is 5 percent.

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Perceptual and Field Factors Causing Lateral Displacement

BY THE TRAFFIC SYSTEMS RESEARCH DIVISION BUREAU OF PUBLIC ROADS

Reported by [1] RICHARD M. MICHAELS, Chief, Human Factors Research Branch, and LEE W. COZAN, Research Psychologist

An awareness of the existence of the phenomenon of lateral displacement has been extant for many years and much descriptive information has been published on this driving phenomenon. However, the study reported here constitutes what is believed to be a first attempt to determine the process drivers use to locate objects and to define the factors that cause a driver to displace laterally.

The results obtained in a controlled study on a test track showed that drivers locate an object they are overtaking on the basis of the angular velocity of the object. If a driver can detect this lateral movement, he knows that the object cannot be in his path. If there is no such velocity, the object is perceived as an obstruction and the driver must displace. The results also provide a basis for predicting the effects of lateral displacement on lane width, size of objects near the path of travel—fixed or moving—and the speed of the vehicle.

Findings reported here have many implications related to highway transportation and can provide some criteria for design, speed controls, and roadside developments in plans for future highways and improvement of existing ones.

Introduction

WHEN AN object is placed near the path of a driver, a lateral movement away from the object occurs as the driver approaches. The amount of this lateral displacement has been shown to be directly dependent upon the distance of the object from the path of travel (*1*, *2*).[2] Thus, Taragin (*2*) has shown that there is a shift in position for objects located up to 6 feet to the right of the driver's path of travel. However, the process that the human operator must carry out in order to locate himself relative to fixed objects in his path has not been specified. The research reported here was an attempt to isolate the variables involved in this location process. Two models were considered; one fits the results obtained in the tests.

From a perceptual standpoint, the transverse location of an object in a driver's path may be considered a problem in trigonometry. The transverse distance a (the lateral distance of an object from the driver) may be derived from the simple trigonometric expression, in which ℓ = location, as:

$$a = \ell \tan \theta$$

These conditions are shown in figure 1. Thus, at any point in space, the observer may determine the transverse distance a by estimating both ℓ and θ. For small angles tan $\theta = \theta$, and therefore the equation becomes simply $a = \ell\theta$.

However, a problem arises for the driver because of the interaction of distance and angle. At long distances the angle is so small that errors in estimation preclude a solution of sufficient accuracy to determine whether the object is in the driver's path. Similarly, at short distances the angle increases so rapidly that solutions also become inaccurate. Therefore, there should be a range of distance for which drivers judgment of the angle θ is most nearly accurate. On the basis of this angle estimation model, as the driver approaches the object, he eventually moves into an optimum range of discrimination. If the angle is smaller than some critical value, the driver will displace away from the object, the magnitude of the lateral displacement being directly related to the size of the angle at the distance at which the detection of the object is made. According to this model, lateral displacement should begin at some fixed distance from the object, independent of the absolute location of the object and independent of travel speed.

An alternative model exists, however. As the driver is moving continuously toward the object, the angle as well as distance is changing continuously. If the driver tracks the object over a period of time and estimates the rate at which the angle is changing, he also can determine the lateral location of the object in relation to his path of travel. The rate of change of the angle is a nonlinear function of time and is, furthermore, dependent upon the speed of travel. If the driver were to operate on this basis, he would be solving the equation:

$$\frac{d\theta}{dt} = \frac{av}{a^2 + \ell^2}$$

Where,

$\frac{d\theta}{dt}$ = rate of change of angle
θ = angle
a = transverse distance
v = velocity of vehicle
ℓ = location of object.

Estimation of the rate of change of the angle between himself and the object in his path has several advantages for the driver. First, his judgment very quickly becomes a simple binary one. If the rate of change does not exceed a critical value regardless of sight distance and object location, the driver can predict a collision course. Second, the driver has a physical anchor for speed judgment and one source of error may be minimized. Third, vehicle speed must be taken into account in any steering inputs imposed upon the vehicle.

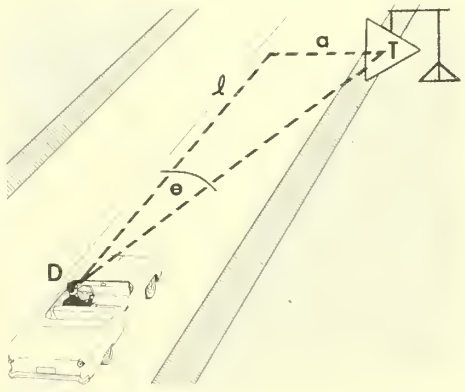

Figure 1.—Geometry of the lateral displacement effect.

[1] Presented at the 42d annual meeting of the Highway Research Board, Washington, D.C., January 1963.

[2] References indicated by italic numbers in parentheses are listed on p. 240.

On the basis of the derivative model a set of hypotheses very different from the angle estimation model may be stated, as: (1) The magnitude of lateral displacement will be directly related to vehicle speed. (2) Lateral displacement will begin at a distance dependent on vehicle speed. (3) The derivative of the visual angle at the point where displacement begins will be independent of speed and object location so long as displacement occurs.

A final consideration that exists in the displacement effect concerns the spatial characteristics of the stimulus object. In the description of both models, it was implicitly assumed that the object was a point in space that served as a simple visual reference. Actually, all practically realized displacing objects have some extension. It would appear reasonable that the nature of the contours of the object would influence the driver's perception of the location of the object. The study of Case et al. (*1*) showed that the size of the object significantly affected displacement.

It might be expected that the angle would be taken to the contour of the object nearest the path of travel. If, however, the shape of the object is of limited extent and has one dominant contour, the driver might be expected to use that as a point of reference; for example, a triangular object having the base oriented perpendicular to the driver's line of vision. It may be expected that when that base is farthest from the roadway (the apex being nearest the travel path), there should be less lateral displacement than when the situation is reversed. Obviously, such an example would be a limited situation for there should be a limit to contour effectiveness if the farthermost border has too great an extent. Within these limits it is reasonable to hypothesize that the dominant figure contours should influence the magnitude of lateral displacement. In the study reported here an equilateral triangle was used to test this hypothesis. In summary, this study was an attempt (1) to isolate the perceptual variables that cause lateral displacement and (2) to discriminate between two alternative models of that process.

Summary

The study reported in this article was made as an attempt to analyze some of the underlying factors that cause the lateral displacement of a vehicle away from a roadside object. The investigation was conducted in daylight and under free field conditions. For several conditions of object location and vehicle speed, the lateral position of the vehicle was measured continuously over a 5,000-foot specially prepared test track.

The findings indicate that lateral displacement is a special case in the field of visual velocity perception. Relative to the observer, the displacing object effectively moves laterally across the retina with a definable angular velocity. Drivers react to this apparent velocity of the object by determining when and how much they should displace on the basis of the time and distance at which that velocity exceeds threshold.

The angular velocity model allows an understanding of the effect of the location of roadside objects on highway capacity as well as lateral displacement. In addition, the results suggest that the limitations in driver's judgment of location of objects may markedly increase the probability of certain types of collisions under conditions of low illumination and headlight glare.

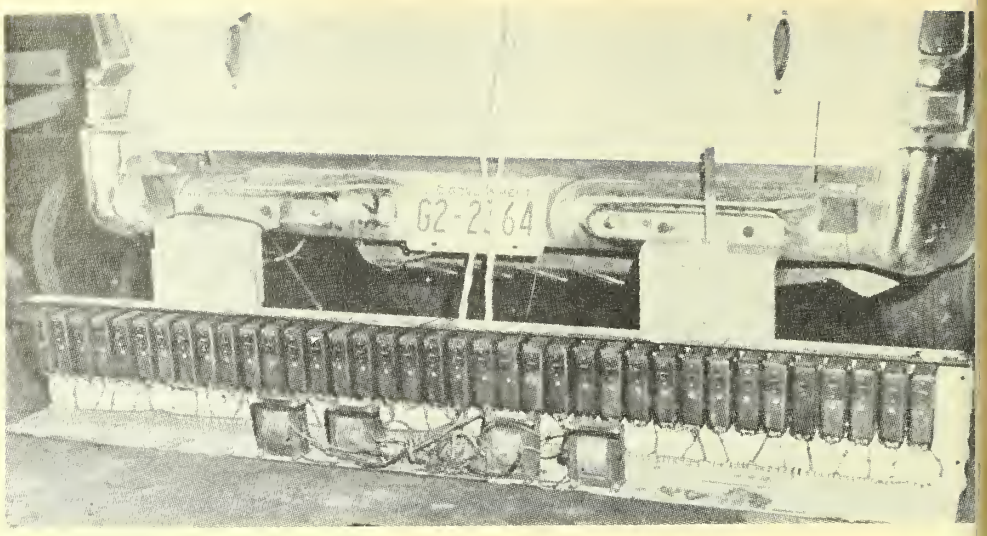

Figure 2.—Lateral displacement detector.

Apparatus and Procedure

In order to determine where and when lateral displacement began and its magnitude, it was necessary to devise a method for measuring lateral position continuously. An optical tracking system was developed by Melpar, Inc., for this purpose. It was a housing, anchored on the rear bumper of a vehicle, that contained 37 individual photodetector units mounted to face downward. The detector is shown in figure 2. Each unit contained (1) a light source and lens system to focus the beam on the roadway; and (2) a mirror system that focused light, reflected from a specially prepared road, on a photoresistor. A schematic of the detector unit is shown in figure 3. To get sufficient light reflected to the photoresistor, a 2-inch retroreflective strip was placed on the pavement. With this material, a high proportion of the light from the lamp was reflected into the mirror and hence to the photoresistor.

The photoresistor was connected directly to a transistor amplifier. When the light reflected onto the photoresistor was high, its resistance dropped and sufficient current flowed to close a relay. Thus, whenever one of the detector units passed over the reflective line that detector, and only it, would fire. As the vehicles were moved laterally, a different detector unit was activated. Lateral position could be estimated to the nearest inch as the units were on 2-inch centers. With use of 37 detector units, lateral displacement could be measured over a length of 6 feet.

To record the lateral displacement dat continuously, the digital output of the ampl fier relays was used to switch an appropriat step in a 37-section potentiometer. Th analog voltage was then recorded on a Brus recorder. With this complete system it wa possible to continuously plot the path of vehicle as it traveled down the test track. B leaving 5-foot gaps in the reflective line ever 100 feet, it was possible to determine later placement as a function of distance from th displacing object.

Test track

The test track was a 1-mile section of a je aircraft runway. The concrete runway wa 100 feet wide and had an asphalt shoulder 2 feet wide on each side. The runway wa made up of four sections of concrete each 2 feet by 20 feet. The maximum vertical curva ture of the test section was less than 0.1 pe cent. A single section nearest the edge of th runway was used. Thus, the travel path wa a lane 25 feet wide having limits demarcate by the asphalt shoulder on the driver's righ and the longitudinal joint on his left. Th reflective strip was laid in the center of th lane. It was placed so accurately that th deviation from the center was never more th 1 inch over the mile course. The reflecti material was a buff color that was clear visible to the observer. No way was foun to camouflage this line and still retain suf cient retroreflectivity to ensure reliable oper tion of the placement detector. The arrang ment of the test situation is shown in figure

Two identical equilateral triangles, 6 fe on a side, and mounted on a boom, were use as the displacing objects. This boom was 1 feet long, a length sufficient to minimize an effect that the mounting base might have o displacement. The boom could be moved or out, and the triangle could be rotated abou its mounting point so that either the base o the apex could be placed nearest the path o travel. One object was placed 2,000 feet an the other 4,000 feet from the beginning of th course.

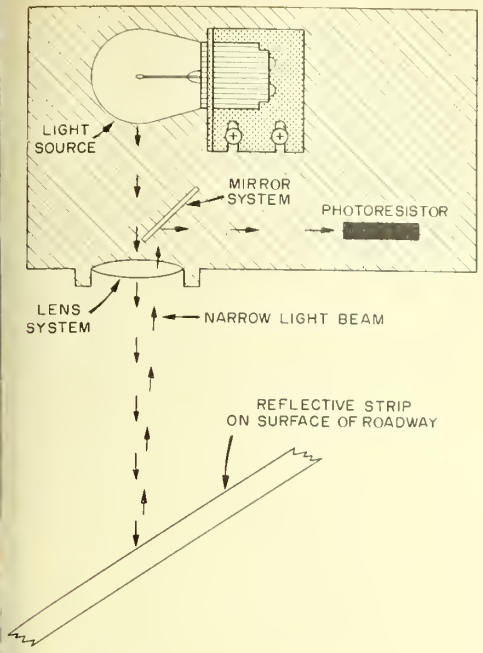

Figure 3.—Photodetector unit.

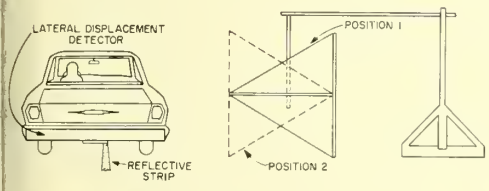

Figure 4.—Road test arrangement.

Four lateral locations for each object were selected. From an analysis of the angle estimation model, the distance at which the tangent function began to exhibit an obvious change in slope was in the order of 200 feet. This model predicts a direct relation between lateral displacement and the size of the angle, hence object location was chosen in units of angular separation at the distance of 200 feet. The closest location was chosen at this point to subtend an angle of 2 degrees. Three other positions were chosen so that they subtended angles of 2¼, 2½, and 2¾ degrees, respectively. In lineal distance, the object was placed 7.0 feet, 7.8 feet, 8.9 feet, and 9.6 feet from the driver. In these experiments, each object was placed at random in one of these four locations. The orientation of the triangle also was arranged randomly. Orthogonal latin squares were used for the complete matrix of test conditions so that any interactive effects between the two displacing triangles were counterbalanced.

Four vehicle speeds were used—15, 30, 45, and 60 m.p.h. Each test driver was tested at each speed for all combinations of object location and orientation. In total, each driver went through a $4\times4\times2$ factorial design. In addition, the design was replicated four times.

All the electronic equipment for measuring and recording lateral position, including a 1.5-kv. generator, was installed in a station wagon. A driver and the experimenter were the only occupants of the vehicle. Four assistants to the experimenter adjusted the position and orientation of the displacing objects according to a prearranged schedule. Test drivers were five men whose ages ranged from 25 to 40 years, all were licensed drivers, each having had 5 years or more driving experience. None was told the purpose of the tests. Rather, the drivers were told that the study was aimed at finding out how well each could maintain the vehicle at a constant assigned speed.

Table 1.—Analysis of variance for lateral displacement under 32 experimental conditions

Source of variation	Sum of squares	*d.f.*	Mean square	*F*, ratio
Vehicle speed (v)	2,911.7	3	970.5	[1] 46.88
Object distance (d)	3,658.4	3	1,291.3	[1] 62.38
Object orientation (o)	252.5	1	252.5	[1] 12.20
Driver (D)	9,610.8	4	2,402.7	[1] 116.07
$v\times d$	110.5	9	12.3	------
$v\times o$	135.0	3	45.0	2.13
$v\times D$	2,015.9	12	168.0	[1] 8.12
$d\times o$	240.1	3	80.0	[1] 3.86
$d\times D$	1,979.4	12	164.9	[1] 7.97
$o\times D$	34.9	4	8.7	------
$v\times d\times o$	226.2	9	25.1	1.21
$v\times d\times D$	1,007.2	36	279.8	[1] 13.52
$v\times o\times D$	297.4	12	24.8	------
$d\times o\times D$	637.6	12	53.1	2.06
$v\times d\times o\times D$	796.4	36	22.1	------
Error (within treatments)	9,917.8	480	20.7	------
TOTAL	33,831.8	639	-------	------

[1] Significant with probability less than 0.01.

Table 2.—Analysis of variance for angular change

Source of variation	Sum of squares	*d.f.*	Mean square	*F*, ratio
Between vehicle speeds (v)	17.3	2	8.65	4.62
Between drivers (D)	7.1	4	1.78	----
Interaction: $v\times D$	34.9	8	4.36	2.33
Error (within treatments)	56.2	30	1.87	----
Total	115.5	44	----	----

Results

The maximum lateral displacement was determined for each condition and each driver. These data were subjected to an analysis of variance and the summary is shown in table 1. Differences among the main variables are significant at the 0.01 level. The analysis also shows a significant interaction among these variables.

Figure 5 is a plot of displacement as a function of object location for each of the four speeds; these curves shown include data for the base orientation only. The line shown is the mean displacement for the five subjects. The general form of the curve is the same as that for each individual driver. The straight-line relationship shown is similar to Taragin's data (*2*), but the magnitude of lateral displacement is less. The displacement at each object location increased markedly as speed was increased; this is summarized in figure 6. Again, the four curves are for the base orientation only. The data demonstrate that lateral displacement is directly dependent on travel speed as well as object location.

In the rate of change of angle model, it was hypothesized that lateral displacement would begin at a distance from the object that was directly dependent upon vehicle speed. Figure 7 shows the relation between vehicle speed and the distance at which displacement began. The parameter is the lateral location of the displacing object. The beginning point ranged from approximately 50 feet at 15 m.p.h. to about 275 feet at 60 m.p.h. The data were consistent in showing a significant increase in distance at which displacement started in relation to increase in speed for all four object locations; thus the hypothesis was confirmed.

Test of rate of change of angle

One of the hypotheses derived from the angular change model was that the rate of change of angle at which displacement began would be independent of both object location and vehicle speed. To test this hypothesis, it was necessary to determine the point at which lateral displacement began for each run. This determination was confounded by two factors. First, there was some variability in lateral position for all drivers. Considerable error was possible in judging the start of displacement as it frequently was difficult to determine whether change in position was made in response to the displacing object or was only a random change in position. Second, not all conditions caused a significant displacement and for these conditions no determination of starting distance was possible. This situation occurred when the displacing object was located farthest from the travel path and the test was made at the lowest speed, at 15 m.p.h. In general, lateral displacement occurred reliably for the three highest speeds and the three closest object locations. The distance at which displacement began could reliably be estimated for these conditions. Further analysis was done only on these data. For these combinations of speed and lateral location of the object, the rate of change of angle was determined for each speed and each driver, and an analysis of variance was done on these data; the summary is shown in table 2. As shown, none of the differences were significant. It does seem reasonable to conclude, therefore, that there is a constant rate of change of angle between the driver and the object at the point where displacement is begun.

The final result of this investigation concerns the spatial relations between the contours of the displacing objects. It was hypothesized that displacement would be greater when the base of the triangle was nearest the path of travel than when the apex was so located. The analysis of variance in table 1 shows that there was a significant difference in lateral displacement related to object orientation. In figure 8, displacement is plotted as a function object location for each orientation and for each speed. As may be seen, the difference in displacement related to the base and apex orientation increased with speed of travel and decreased as object

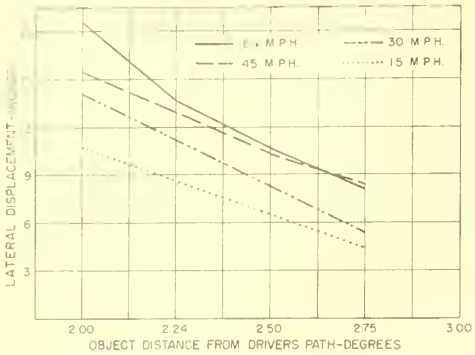

Figure 5.—Lateral displacement as a function of object location at different speeds.

distance increased. Again, the results offered verification for the hypothesis that lateral displacement should be dependent upon the geometric characteristics of the displacing object.

Discussion

The data clearly indicate that a model of lateral displacement based on the rate of change of visual angle best fits the obtained results. The three hypotheses originally specified for this model were validated. Thus, as the model predicts for one hypothesis, a direct relationship between the magnitude of lateral displacement and travel speed was confirmed. A second hypothesis, that lateral displacement would be started at a longitudinal distance functionally related to vehicle speed, was also confirmed by the data. The data for the third hypothesis, that the determining factor in lateral displacement would be constant over all conditions, indicated a strong confirmation.

Threshold for visual velocity

Thus, the study leads to an explanation of lateral displacement that is based upon the driver's ability to detect the rate of change of visual angle of objects near his path of travel. The problem for a driver approaching an object near his path of travel is one, from a perceptual standpoint, in which, phenomenally, the image of the object moves across the retina. This movement is actually a special case in the general field of the visual perception of velocity. The major differences are that (1) the angular velocity of the target in the driving situation is nonlinear, and (2) the visual angle subtended by the object itself increases as the observer approaches.

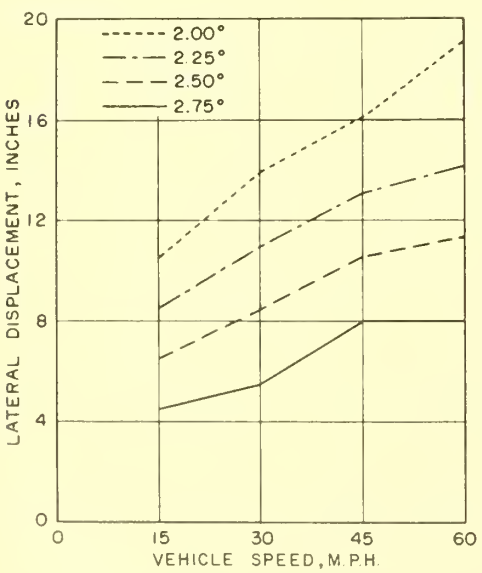

Figure 6.—Lateral displacement as a function of vehicle speed for different object locations.

From this viewpoint, it is worthwhile to compare the angular velocity at which displacement begins with the classical research on the threshold for visual velocity. The work of J. F. Brown (*3*) indicated absolute thresholds in the range of 1.0 to 10.0 minutes of arc per second, and the more recent work by Rock (*4*) indicated an absolute threshold range of 0.2 to 0.5 minute of arc per second when luminance was carefully controlled. In the present experiment the range of angular velocity at the beginning of displacement was from 4 to 40 minutes of arc per second. It is obvious, therefore, that the driver was responding to the presence of an object near his path of travel at a point where its angular velocity approached his absolute threshold.

Within the framework of this model, it is possible to define the process of displacement. If the driver, traveling at a certain speed, increases his fixation distance along the roadway two things occur:

One, the angular velocity of elements in his field of view decreases rapidly. Eventually all elements become subthreshold regardless of their lateral separation.

Two, objects located at increasing lateral separation from the line of sight fall on the eye outside the driver's fovea centralis,[3] which will cause a sharp decrease in sensitivity to velocity. Thus, there exists a visual operating field whose size is determined by a physiological characteristic of the eye and a physical function. This visual operating field is shown in a slightly different way in figure 9. As the driver increases the distance ahead at which he establishes a point of reference, as shown on the abscissa, the lateral distance at which an object must be located in order to generate a detectable velocity increases rapidly. Assuming the visual field is 3 degrees at 60 m.p.h., at a linear distance of 300 feet, objects located laterally more than 16 feet from the driver's path are outside his visual field. Conversely, all objects laterally less than 14 feet from the driver, although within the operating field, have subthreshold angular velocity at this distance. Actually, it is only the lateral locations shown within the hatched area of figure 9 that have a detectable angular velocity at 60 m.p.h. Thus, as a driver approaches an object that is within his visual operating field at 60 m.p.h. at a distance of about 300 linear feet, if there is no detectable component of angular velocity the object will appear to be in his path, and he will begin to displace laterally.

[3] The small area in the eye where form and size discrimination are best.

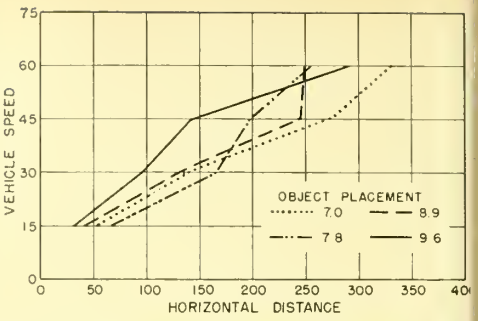

Figure 7.—Effect of speed on distance from object that lateral displacement is started.

The process for detecting lateral positio becomes a fairly direct one for the driver. H must adjust his point of fixation to that dis tance at which there is a sharp decrease i angular velocity for objects at the margins o his visual field. He can determine this poin from a variety of cues in the driving enviro ments such as pavement texture, shoulde contrast, etc. As obstructions first enter hi visual field, the driver is able to make a simpl binary judgment. If the obstruction has detectable lateral movement it cannot be in h path, and no displacement is necessary. If has no detectable lateral velocity it is locate in his path and hence he begins to displace.

The foregoing considerations indicate th the driver has a very small margin of time an distance within which to operate on object located laterally along the path of trave Assuming no restrictions in sight distanc only 3 to 10 seconds are available for th driver's decision as to whether a displacemer is necessary and how much is required. B operating near the absolute threshold angular velocity, the driver not only has stable reference for detection but also max mum time for object location, as well maximum time for making compensato steering responses.

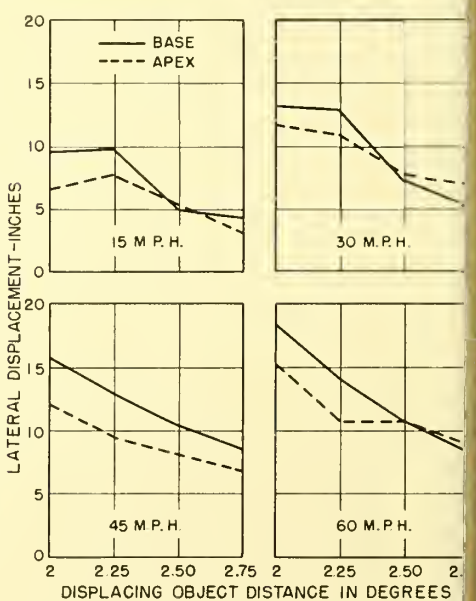

Figure 8.—Lateral displacement for diffe ent object orientation for each obje distance and for each speed.

It appears reasonable to expect that those factors found from classical research to influence the perception of visual velocity would be applicable to lateral displacement. Thus, the object size may be expected to influence displacement because of the effect of stimulus size on visual velocity (*3*). This factor of size as it affects lateral displacement has been studied by Case et al. (*1*). They found that there was a significant effect on the displacement starting point and also the magnitude of the displacement as a function of the displacing object's size.

It may also be expected that the shape of the stimulus will influence the visual perception of velocity. The results of this study demonstrated that there was a significant reduction in lateral displacement, approximately 15 percent, when the apex of the triangle was oriented toward the driver's path of travel. Phenomenally, of course, these results imply that the apex-oriented object has a higher visual velocity than does the base-oriented one. The higher the speed of travel the less will be displacement as displacement occurs in relationship to perceived velocity of the displacing object in this model.

Effect of shape

The effect of shape has been studied by Motokawa (*5*) by means of electrical stimulation of the eye. His findings bear directly upon the effects on lateral displacement found in this investigation relative to the triangular displacing object orientation. His work suggests that the physiological correlate of visual velocity is the amount of suppression of retinal response exerted on the retinal pathway through which the image of the moving object has passed. This concept, called retrograde suppression, can account for most of the perceptual results in the study of visual velocity. Thus, Motokawa suggests that as a moving stimulus passes across the retina, a field is generated about the object that suppresses activity in the area removed from the immediate vicinity of the stimulus itself. Thus, as a stimulus moves across the retina it generates retinal activity as it proceeds and acts to extinguish or neutralize the retinal activity in the path through which it has already passed. Hence, the lower the velocity the more intense the stimulus, or the larger the object the greater will be the degree of retinal suppression, both causing a perception of lower angular velocity. In essence, the strength of the suppressing stimulus is the correlate of the perception of velocity.

The intensity of the suppression is also related to the nature of the contours of the stimulus. Other experimentation by Motokawa (*6*) has shown that the strength of the field about an object is determined by the contours of that figure as well as its size and brightness. For a triangle, as used in the displacement study reported here, the field of activity is at a minimum at the intersection of the figure contours. Consequently, the strength of the field that acts as a suppressor on trace activity in the retina is at a minimum. The perceived velocity of the figure will be at a maximum when the apex of triangle is oriented to path of travel. It is, then, on the basis of the differences in fields of suppression that the reduction in lateral displacement obtained in this study can be explained when the apex is oriented closest to the driver's path of travel.

APPLICATIONS

Placement of Objects

In the light of this study, it is instructive to examine current practices relative to the location of signs and abutments near the roadway. The *AASHO Manual for Signing and Pavement Marking of the National System of Interstate and Defense Highways, 1961*, requires that no object be placed nearer than 6 feet to the travel lane. At this separation, as may be seen in figure 9, an object will have a detectable angular velocity depending on the driver's visual threshold for distances up to 300 feet ahead of a driver traveling at 60 m.p.h. As the driver's reference distance at this speed is slightly less than 300 feet, such a standard ensures that all objects will have a suprathreshold velocity and hence the driver will locate them outside his travel path. Consequently there will be no displacement.

It is also obvious that this standard is applicable only to highways on which the travel speed averages 60 m.p.h. At higher speeds a greater separation would be required, and at lower speeds a closer spacing may be tolerated. As a matter of fact, the data from this study allow the determination of minimum placement of roadside objects for any desired travel speed. The curve in figure 10 shows this relation for a 12-foot lane and indicates that the minimum lateral location to eliminate displacement increases continuously as travel speed increases. It is also obvious that at low speeds objects actually encroaching a 12-foot travel lane may be tolerated by the driver without his making a lateral displacement.

Every attempt was made to obtain a maximum lateral displacement in the design of this study. It was initially anticipated that the magnitude of displacement in this study would exceed that obtained by Case et al. (*3*) or Taragin (*4*) because an effective 25-foot lane width was employed and no other obstacles were in the driver's path. This prediction was not borne out in the study. Actually, the magnitudes of displacement were a half to a third less than reported in the other field studies. Two reasons may account for this unexpected result. One is in the nature of the displacing objects and the other is the factors affecting the driver's ability to judge his line of travel.

In this study the absolute size of the object was 15 square feet; this object was considerably smaller than the displacing objects used by Case et al. (*3*), which had minimum and maximum sizes of 28 and 64 square feet, respectively. In Taragin's study (*4*) two of the objects were considerably larger than the triangles used in this experiment. In terms of the model of displacement proposed in this article, it would be expected that the apparent velocity of the displacing object will be greater

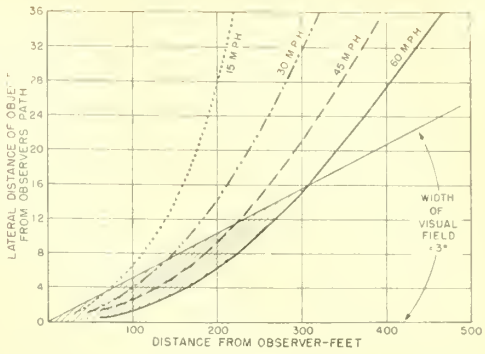

Figure 9.—Visual velocity threshold contours at four speeds.

for the smaller object and hence appear farther from the driver's path of travel.

In relation to the factors influencing the driver's ability to judge his line of travel accurately, in this study the reflective strip that was placed on the pavement to measure lateral position was clearly visible to the driver. All five drivers appeared to orient themselves relative to this marking so that it was nearly centered under the vehicle. The striping apparently served as a direct reference by which the driver could define his path of travel. By having a stable reference at which the driver may fixate, the detection of movement of an object near his projected path should be improved. With no such reference for fixation, the driver's line of vision may be expected to vary laterally. This should reduce the accuracy of his estimation of the apparent velocity of the object and hence add ambiguity about the judgment of object location. It seems reasonable that such uncertainty would amplify a driver's response to the displacing object, and the result would be a greater magnitude of displacement. If this explanation is valid, it should be possible to reduce the magnitude of lateral displacement in a field situation by providing a tracking reference line for the driver. A test of such an hypothesis is currently underway at the Bureau of Public Roads.

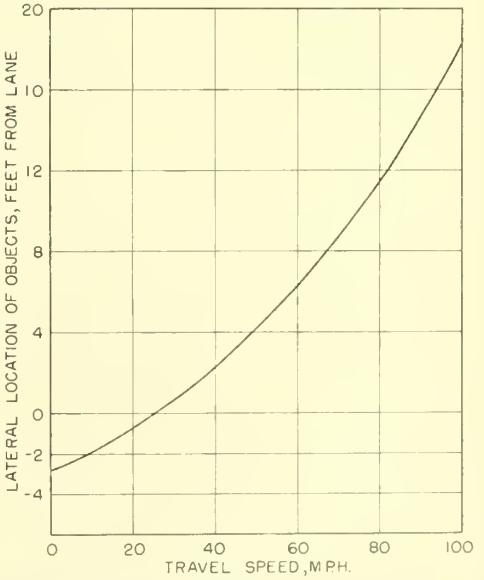

Figure 10.—Nearest placement of roadside objects that causes no lateral displacement.

Speed Control

From this study, it is clear that a driver can exercise control over the magnitude of his lateral displacements by reducing vehicle speed. Conversely, it should be possible to cause a speed reduction by strategically placing objects relative to a travel path within which the driver has little freedom to displace. Such situations occur in construction areas and special channelization situations. From the data presented here it is possible to develop a relation between lateral location of objects near the path of travel and the travel speed through the section.

Three conditions must be met in order to control speed in this fashion. First, there must be no possibility of shifting from the travel lane. Second, a desired terminal speed must be selected. Third, a maximum acceptable deceleration must be specified. The last two determine the length of the speed transition zone. For example, if the input speed is 50 m.p.h. and it is desired to reduce speed to 15 m.p.h. and have deceleration not to exceed 1 m.p.h. per second, then approximately a 1,100-foot transition section must be used. Given these three conditions it is possible to use the two curves shown in figure 11 to determine the lateral location of the objects (curve 1) and their separation from one another (curve 2).

The use of these curves may be shown by an example. Suppose that travel speed approaching a construction zone is 50 m.p.h. and that it is desired to reduce the speed of approaching traffic in a 12-foot lane to 15 m.p.h. and have a deceleration not to exceed 2 m.p.h. per second. Assuming an initial sight distance greater than 500 feet, a cone would be placed relative to the lane so that the driver would decrease his speed over this 500 feet to 40 m.p.h. Curve 1 in figure 11 specifies this placement relative to the driver at approximately 11 feet. Assuming the approaching vehicle is located in the left of the lane, then the cone should be placed 1 foot in from the edge of the lane. The next cone should be located at a distance from the first as determined from curve 2, also for 40 m.p.h. This requires that the second cone be placed 180 feet beyond the first and one-half foot from the edge of the lane. In order to cause a speed reduction to 30 m.p.h., a third cone should be placed 3½ feet from the edge of the lane, 140 feet from the second cone, this should then be repeated and a fourth cone placed 140 feet beyond. To reduce speed to 20 m.p.h. the lateral position for the next cone, determined from curve 1, should be 6 feet from the driver or 4½ feet in from the edge of the lane. The fifth cone should be placed, as shown in curve 2, 85 feet beyond the fourth cone. A sixth cone repeats the spacing for the fifth cone 85 feet beyond the fifth. Finally to reduce speed to 15 m.p.h., a seventh cone should be placed 5½ feet in from the edge of the lane 70 feet beyond the sixth cone.

Thus, by using a minimum of seven cones placed as described, a smooth reduction of speed can be brought about as a natural consequence of the placement of objects in the travel lane. Obviously, more cones could be used, being placed according to the relation with speed shown in curve 1 of figure 11. This approach to traffic speed control would offer several advantages over existing techniques if it actually holds in the field situation. It would allow a specific definition and prediction of the speed of traffic and would ensure a smooth speed transition. It may also permit a far more reliable means of control than can possibly be obtained with construction signing. Hence, this approach could lead to safer traffic movement through a hazardous area.

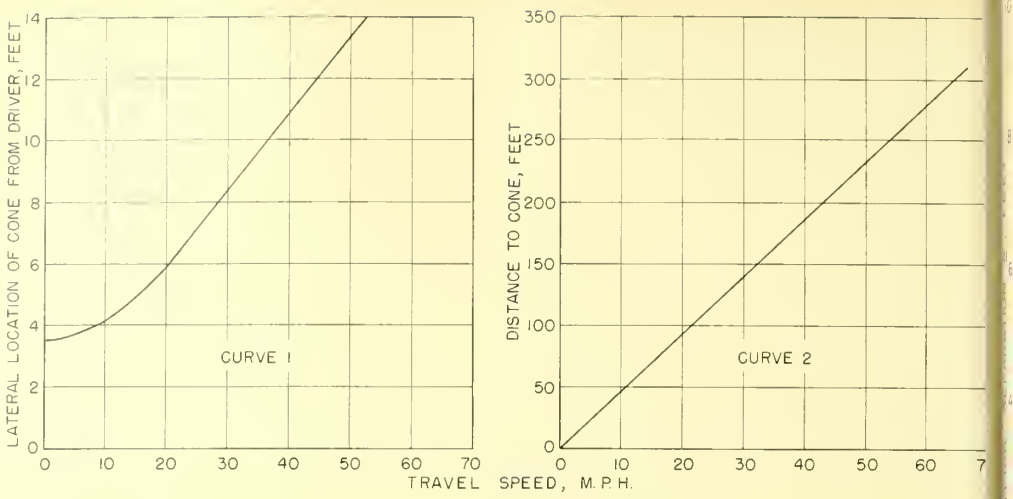

Figure 11.—Placement and spacing functions of objects for speed reduction.

Effect of Truck Width on Traffic

The effect of vehicle width on traffic has received considerable discussion. One aspect concerns the influence of trucks on traffic and safety. The problem essentially is one of definition of how the size of a vehicle influences traffic, if it does. One criterion of influence that may be used is: When two vehicles approach, neither shall by its presence in its own lane cause the other to change its placement in its lane. Such a criterion has three advantages: (1) Forced changes in placement of a stream of traffic may be expected to cause turbulence in flow and hence affect the efficiency of traffic movement. (2) A shift in placement forced on a driver implies that the driver predicted himself on a collision course. This may be perceived by such a driver as an unsafe situation. (3) It is possible to operationally define such a criterion.

Using this criterion, the problem basically is a variation of the lateral displacement effect. It may be treated as if one vehicle were a fixed object on the driver's left and the other vehicle were overtaking at a velocity (v) equal to the sum of the two speeds. Any lateral shift under these conditions will be caused by the driver's perception that the angular velocity of the oncoming vehicle is below threshold. From the data in this study, the factor influencing this perception is the speed of the affected vehicle, which determines the length of the driver's field of view. Thus, to calculate the influence of a vehicle B on another vehicle A, it is necessary to determine whether the angular velocity fo driver of A exceeds the threshold for the partic ular conditions of v_A, v_A+v_B, and latera separation. By using the threshold value fo angular velocity developed in this study, i is possible to derive the relation betwee lateral separation of the two vehicles and th combined velocity v_A+v_B for different value of v_A. This is shown in figure 12. In thi set of curves, the ordinate is the lateral separa tion, the abscissa is the combined speed, an the parameter is the speed of the influence vehicle, v_A. For each curve, all points lyin to the right of each v_A curve generate a supra threshold angular velocity and hence will no cause a displacement.

Example

As an example, assume a lateral separatio equal to 4 feet, and v_A equal to 60 m.p.h At what combined speed, v_A+v_B, will there b no influence of B on A? Using the 60 m.p.h curve, find the point at which lateral separa tion of 4 feet intersects this speed curve From the abscissa, the combined speed ($v_{AB}+v_B$) required to generate a supra-threshol velocity is 184 feet per second (f.p.s.). A v_A is known to be 88 f.p.s., v_B must be 96 f.p.s In other words, the speed of the influencin vehicle, B, must be 64 m.p.h. or more i vehicle B is *not* to influence vehicle A. Con versely if v_B is actually less than 64 m.p.h. B will not generate a threshold angular velocity hence vehicle B will be perceived in th path of the approaching driver A who wil displace to the right.

From data on vehicle placement as a func tion of lane width (7), it is possible to specify the conditions that exist on a highway wher one vehicle overtakes another. The data show that 90 percent of the time, separations from trucks that are 96 inches wide will be no less than 4 feet. For trucks 102 inches wide, it may be expected that separations would be no less than 3.5 feet. If the 85th centile speed on 2-lane rural highways is about 60 m.p.h., it is possible to determine the effect of these two different truck widths on opposing traffic on roads of different lane widths. This is shown in figure 13. It

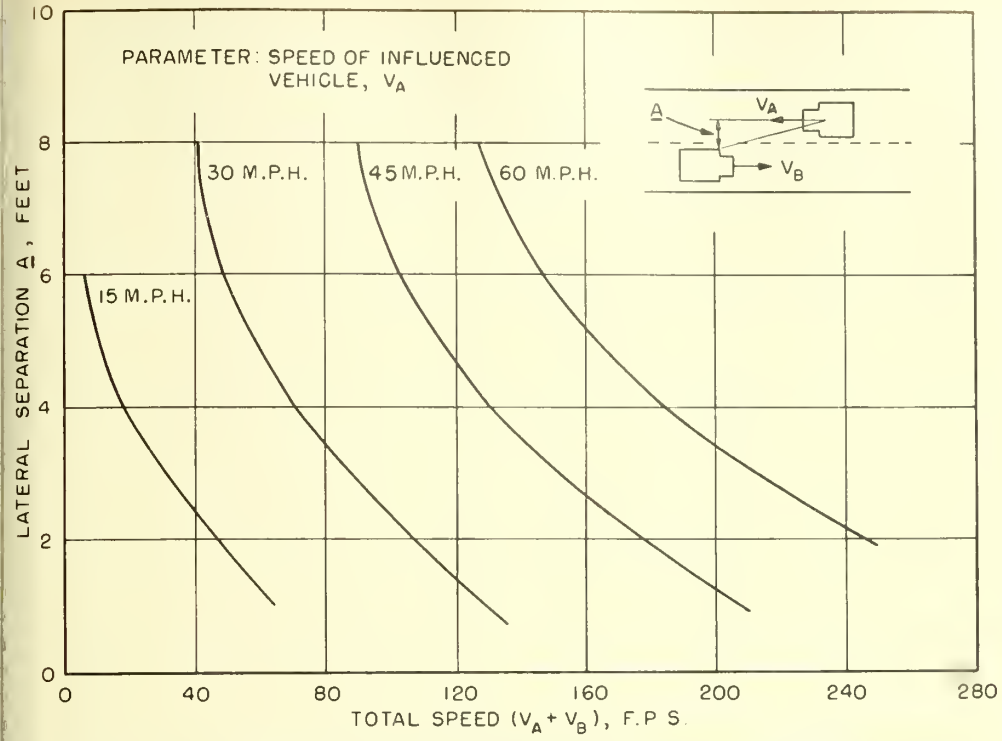

Figure 12.—Lateral separation and speed functions that influence approaching vehicles.

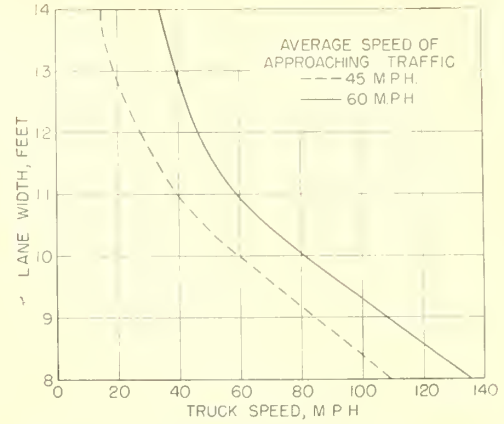

Figure 13.—Minimum speed of a 96-inch wide truck that has no influence on approaching vehicles as a function of lane width.

may be seen that with a 12-foot lane width the 96-inch wide vehicle must, at the 4-foot separation, be traveling at a speed in excess of 64 m.p.h. if it is not to influence opposing traffic. For the 102-inch wide truck, its speed must exceed 74 m.p.h. at the 3½-foot separation to ensure no displacement effect on the opposing traffic. From figure 13, it is possible to determine these relationships for any combination of speed and lane width.

The two factors of speed and lane width in relation to width of truck are the main influences on magnitude of displacement. Influence on traffic varies inversely with the speed of the truck and directly with the speed of the approaching vehicle. In general, truck speeds on main rural highways are significantly lower than that of passenger cars. At higher speeds of the approaching vehicles, truck speed must be nearly the same or higher to avoid causing displacement of the oncoming vehicles. The less the initial separation the greater must be the truck speed. Although truck speeds have increased, keeping pace with increased passenger car speed, it is apparent that as the speed trend of passenger cars increases, a constant difference in speed between the truck and passenger-car will cause ever-increasing displacement of the passenger car approaching the truck. Increased truck width will markedly increase this cross-stream effect and to an ever greater extent as traffic speed rises.

The second aspect is related to lane width. Lane width is one determinant of placement and hence a determinant of lateral separation between opposing streams of traffic. The narrower the lane, the greater must be the cross-stream displacing effects. This may be seen in figure 13. Here the mean placement of the opposing streams in their lanes is used. For example, on a 10-foot lane, a 96-inch truck would have to travel 81 m.p.h. to ensure no displacement, and a 102-inch truck would have to travel at 94 m.p.h. when the approaching cars have an average traffic speed of 60 m.p.h. At a 45 m.p.h. average speed for approaching cars, the speeds of the 96-inch and 102-inch trucks must be 59 and 71 m.p.h., respectively. Thus, small changes in lane width or truck width generate large changes in cross-traffic effects.

This analysis has not considered several variables that can influence the effects of truck width, such as the sight distance, other traffic, or pavement markings. The last of these especially should act to reduce these cross-stream effects. Pavement markings provide a significant additional cue to the driver for locating opposing traffic. Indeed, a center line marking allows a driver to use an additional perceptual process for judgment of position. He now has available a basis for employing visual acuity or, more specifically, minimum separable acuity for making judgments of position. If the driver can detect a gap between the pavement marking and the nearest contour of the truck, then he knows that the truck cannot be in his path. This cue, of course, may be at variance with the angular velocity cue and what a driver does may depend on the magnitude of the disparity between the two. However, it may be expected that, in general, the added cue of pavement markings will reduce cross-stream effects. This will be tested in future research.

The Effects on Lane Capacity of Shoulder Objects

It is well known that an object located near a travel lane will reduce lane capacity. An understanding of why this happens may be had from the lateral displacement effect shown in the study reported here. As a driver approaches a fixed object he is detecting its location at the farthest point in his visual field at which he can detect angular velocity. The distance at which such detection is made and the magnitude of the displacement are directly dependent on travel speed. If a driver is in a line of traffic traveling at 30–40 m.p.h. and traffic is also approaching, one way he can reduce the magnitude of displacement from a shoulder object is to reduce his speed. A slight reduction in speed will not only give him more time for locating a shoulder object but also will minimize the magnitude of displacement. However, at or near lane capacity, slight reductions of speed must reduce the lane capacity.

In addition, at lane capacity, the headways maintained will cause a leading vehicle to limit the forward view of a following driver. This will markedly influence the angular velocity of shoulder objects as they become visible for they will enter a driver's field of vision at a closer distance than in free flow. If a driver detects the presence of a shoulder object before he can detect its location—as will normally be the case—he may be expected to adopt a greater headway in order to improve his detection of location. This increase in headway will obviously begin considerably before the actual location of the shoulder object. Under these conditions, headways will increase and lane capacities measured at the object will be lower than they would be if there were no object. The presence of a shoulder object will markedly affect the speed-spacing relationships in a queue of traffic and will be most evident at or near lane capacity where displacement must be limited because of traffic in adjacent lanes. To maintain adequate control, the driver is forced to compromise in speed and/or headway. This should lead to a reduction in capacity when the highway is operating at or near capacity. If the changes in speed or headways in a queue are to be detected, measurement of these variables must be made considerably in advance of the object causing them, where will depend upon the visibility of the obstructing object.

Night Visibility and Collision Avoidance

This investigation also has relevance for the more general problem of the visibility of stationary objects on or near the highway. This problem, especially acute at night, has received considerable attention in the safety field for many years. In general, it has been conceived primarily as a problem in object detection. From the results obtained in this study, it would seem reasonable that not only the detection of an object is important but also the ability of the driver to locate that object relative to the path of travel. For, even if the driver detects the object, it is also necessary to ascertain whether it is in or near his path of travel. Obviously, the brightness of the object or its contrast with its surroundings, its size, and its shape, and fundamental factors in this dynamic localizing process. In this context, two classes of collision situations may be considered: collisions with a fixed object and head-on collisions.

In approaching a fixed object in or near the travel lane, a driver is faced with two problems. One is the detection of presence of an obstruction and the other is the detection of its location. The former problem has been studied and many of the determinants of visibility have been defined. It would appear that visibility (detection of the presence of an object in a driver's path) alone cannot account for the frequency of collisions with fixed objects. Only under extreme operating conditions will visibility be so compromised that collisions will occur because drivers do not see an object.

On the other hand, many of the factors that influence visibility also influence localization. These factors of size, shape, contrast, and brightness have as much of an effect on detection of angular velocity as they do on visibility. Thus, for example, from Blackwell's data (*8*) an object 6 feet wide having a contrast with its surroundings of 0.010 at an adapting brightness of 0.1 footlambert would be detected at about 800 feet from the observer. From the data collected in the study reported here, the driver cannot make localization detection at distances much beyond 300 feet for speeds under 60 m.p.h. Hence, it would seem far more reasonable to look at errors in location as a basis for fixed object collisions.

Because of the limitations in localization detection, one obvious prediction is that collisions with fixed objects should increase more than other types of accidents from day to night driving. Data from accidents on main rural highways indicate that the total accident rate doubles from day to night, but the rate for collisions with fixed objects more than triples. It is also predictable that the probability of such collision should rise as the effective size of a fixed object decreases. In this context, effective size refers to the area of an object that is continuously above the threshold of visibility for the approaching driver. This relation would exist because the apparent angular velocity of an object decreases with increasing size. Hence, the smaller the size of the object the greater is its apparent velocity. In terms of localization, this could mean that smaller objects more likely would be located outside the driver's path of travel than would larger ones, all other things being equal. It is reasonable to predict on this basis that highway lighting may reduce collisions with shoulder objects but not those occurring within the traffic stream.

Effective size can obviously be translated into other dimensions of the visual stimulus and the same sort of relations predicted. Thus the brightness of the object or its contrast with its surroundings will all effect localization in expected directions. Glare, which reduces the contrast, will also markedly raise localization thesholds.

Examination of the factors influencing collisions with fixed objects indicates that this problem involves far more than simple detection of the presence of an object. Although accident reports on this type of collision frequently quote the driver as not seeing the object, it would appear that these reports more usually refer to a driver's not seeing the object's position relative to his path of travel. It is apparent, therefore, that any thorough analysis of this problem must be concerned not only with absolute visibility but also with the accuracy of roadside object location and the driver's ability to locate himself in his path of travel.

The problem of head-on collisions may also be viewed within the context of the localization problem. Here the problem for the driver is to locate an approaching vehicle relative to his path of travel. The same factors determining displacement relative to truck size apply here. The conditions are far more severe at night, however, than in daylight for many of the cues present in daytime are eliminated at night. Thus cues to vehicle size are almost completely missing because of headlight glare. The localization cues using pavement markings also are almost eliminated. Thus, at night, a driver must localize approaching vehicles on the basis of the angular velocity contributed by the headlights. In effect, a driver has only a dominating glare source to use as a cue to localization.

Localization judgments are made at distances under about 300 feet. At these distances normal separations between approaching vehicles is 1.6 degrees. On low beams, the effective intensity at an approaching driver's eye is of the order of 17,000 footlamberts. The size of the source is 5 inches. Thus, the approaching driver is faced with an extremely bright source of extremely small size. The two complement to generate a high apparent angular velocity. To the approaching driver this implies a large separation between himself and the vehicle he's approaching.

In addition, the glare from the approaching headlights sharply reduces the cues available to the driver for locating himself in his own lane. Consequently, in the approaching situation, conditions are conducive for a driver making maximum errors in locating an approaching vehicle relative to his own path of travel, which should increase the probability of head-on collisions. Accident data from main rural highways do, in fact, indicate that the rate for this type of collision rises more at night than does any other type of collision except those with fixed objects. Thus, where the rates for angle and rear-end collisions increase about one-half from day to night, head-on collision rates rise 2.5 times. In part, at least, this increase may be attributed to the degradation in localization detection because of the extreme conditions existing in these approaching situations.

One advantage of medians becomes readily apparent from these considerations. Medians effectively eliminate the entire problem of the location of approaching vehicles. From the data in this study, medians of only 5 to 10 feet are needed. However, in darkness, the higher figure will minimize not only errors in detection of location but will also leave some brightness contrast in the median area. That is, there will almost always be some detectable contrast between pavement and median that will improve localization for the driver. It is also obvious that the effects of a wide grass median may be obtained with a fence or glare screen and a narrow separation between opposing lanes. From the standpoint of localization detection, any object such as this will function as an interposition cue that will almost completely eliminate any uncertainty as to location of an approaching vehicle. Thus, either wide grass median or a divider providing an unambiguous cue to separation may be used to prevent localization errors. From the drivers' standpoint either would be effective.

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The Automobile in American Daily Life

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Data pertinent to the ownership and use of the automobile in daily life in America have been compiled from several motor-vehicle-use studies and summarized and analyzed in this article. State motor-vehicle-use studies, a Nationwide Automobile-Use Study conducted by the Bureau of the Census for the Bureau of Public Roads, and other Census and Public Roads statistical studies provided the source material for this article. The analysis presented here is expected to be useful to highway planners in assuring that future highway construction will be adequate for the area to be served.

Although all States did not participate in the motor-vehicle-use studies, the characteristic factors of automobile ownership and use presented in this article may be considered representative of all sections of the United States. This was substantiated by comparison of the results from the State studies with the findings obtained in the national study conducted by the Bureau of the Census. However, when detailed pertinent local information on the characteristics of ownership and use of automobiles and travel is needed to supplement other planning and research work, the basic data must come from a State study such as the motor-vehicle-use study.

Introduction

THE PRIMARY purpose of this article is to present summary data on the characteristics of ownership and use of automobiles [1] based on the findings of a number of studies. Data on ownership and use from studies in 18 States have been added to the results reported in *Motor-Vehicle-Use Studies in Six States* (*1*).[2] Complete reports are not available from all the more recent studies, but sufficient data have been obtained to permit expansion and updating material in the earlier report.

In addition to the State motor-vehicle-use studies, the Bureau of the Census under contract with the Bureau of Public Roads has collected national data on some characteristics of automobile use. These data have been correlated with economic data that Census collects, such as that for income and composition of families.

Information is presented here on distribution of automobile ownership by occupational groups, distribution of drivers by age and sex, mode of travel used for home-to-work trips, and purpose of travel. Data are also presented on automobile ownership by income group and by purpose of travel for each trip for each day of the week. Some comparisons have been made between the data provided in the motor-vehicle-use studies conducted by most of the State highway departments at different times between 1935 and 1940 and the studies conducted since 1951.

[1] The terms "automobile" and "passenger car" are used synonymously in this article.

[2] References indicated by italic numbers in parentheses are listed on p. 255.

Background

No great battery of classified data need be marshaled to support the conclusion that the automobile, no less than the motortruck, has been a dominant shaper of the American way of life. The extent of dependence on the automobile has long been recognized by the Bureau of Public Roads and others; it was stated as long ago as 1949 by Wilfred Owen (*2*) in introducing his study of automotive transportation. He said in part:

"Automotive transportation is the most extensive medium of passenger movement in the United States, dwarfing all other agencies of transport combined. . . . The importance of passenger car transportation in the family budget is . . . indicated by the fact that after food, housing, and clothing, automobile transportation outlays are fourth in order of magnitude among consumer expenditures. . . .

"The effects of the automotive age on the average American may be seen in the location of his home and his work, his occupation, his recreation, and in the enlarging radius of his social and business activities. . . . Today it is difficult to visualize the American economy before the advent of the motor vehicle, so completely has the Nation become geared to the workings of internal combustion.

"There is indication that the far-reaching effects of the motor vehicle on our industry and living habits have only begun to be felt."

The growth and influence of the use of private automobiles has been almost entirely a 20th-century phenomenon, the automobiles registered at the beginning of the century numbered a mere 8,000. In 25 years the number registered had risen to 17 million, in 50 years the number increased to 40 million (*3*), and by 1961 registered automobiles numbered more than 63 million (*4*). The number of registered automobiles is expected to continue to increase and by 1966 total an estimated 75 million and by 1976 reach an estimated 95 million (*5*). In 1961 more than 89 million operator licenses were estimated to be in force (*6*). The advent of the automobile has greatly enlarged the area within which people can live and work. For people who lived 3 miles from the heart of the city in 1890, from 30 to 45 minutes was required to get downtown by a streetcar. The competition between the public transportation systems and private automobiles was noticeable by 1920. People had found that they could live farther from their places of employment and get to work by automobile, even over unpaved roads, in 10 to 15 minutes (*7*). Thus a chain of circumstances was set in motion that has had and continues to have great economic and social implications. One of the more recent effects is the number of outlying shopping areas and medical centers that have been developed to service people living from a few miles to 15 or 20 miles from the central business districts. The evidence suggests that people living in outlying areas are willing to travel farther to get to their places of work than to get to shopping and service areas.

The resultant influx of automobiles into the downtown business areas from the mass home-to-work movement of workers has created problems. State and local governments have been faced with the necessity of building urban expressways. At the same time provisions for parking have been necessary; parking lots and parking meters have been established throughout most business districts.

Data from the motor-vehicle-use studies of 21 States show that 53.7 percent of gainfully employed persons requiring transportation to work drove their automobiles and 14.8 percent were passengers; thus 68.5 percent of this group traveled to work in automobiles. According to table 94 (*8*), "Means of Transportation to Work of Workers During the Census Week, for the United States, Urban and Rural: 1960," automobiles were used for transportation to place of employment by 69 percent of the workers.

State motor-vehicle-use studies

Recognizing the need for up-to-date information on the characteristics of ownership

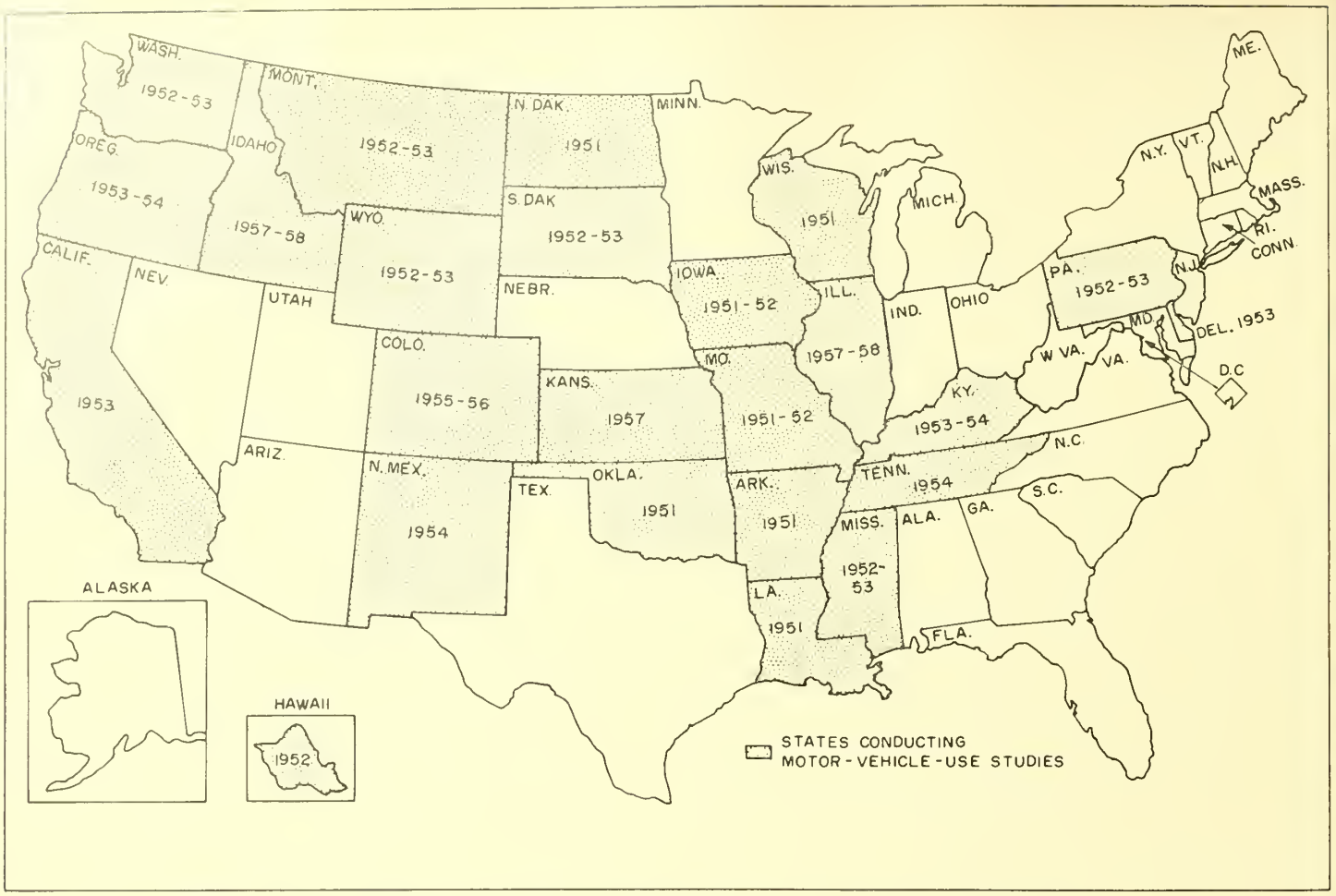

Figure 1.—States participating in the motor-vehicle-use studies and year of study.

and travel by automobiles and trucks, 24 State highway departments in cooperation with the Bureau of Public Roads have conducted State motor-vehicle-use studies. These studies, made at different times between 1951 and 1958, were designed to provide information, not available from regular traffic count surveys, about the characteristics of ownership and use of motor-vehicles, trip-length distributions, occupational groupings of members of households, mode of transportation used for home-to-work trips, purpose of travel, and age and sex of motor-vehicle operators. Data from similar studies conducted by most State highway departments between 1935 and 1940 have permitted historical comparisons to be made by State for the distribution of data on travel for business and pleasure.

Nationwide automobile-use study

In the fall of 1959 and the spring of 1961, the Bureau of the Census, under contract with Public Roads, collected data on a national basis on characteristics of ownership and use of automobiles. Because the Bureau of the Census surveyed the same households (approximately 4,000) from which other economic data had been obtained, the automobile information collected for Public Roads could be correlated with economic information on family income and age composition of the households.

Applications of Data

The results of the 24 State studies and the national study have been used for many different purposes at all levels of government. At the national level, the basic data available from these studies have been used by congressional committees studying highway legislative affairs, by civil defense planning officials, by national and regional planning officials, and by other government agencies. State, city, and county planning agencies have used the travel data on the different highway systems to enable administrators to plan for the future demands for highway transportation. State legislative committees have adapted the motor-vehicle-use data for use in determination of the equity of the present tax structure and proposed revisions. These data have also proved to be useful in developing mathematical models for forecasting traffic.

Information on the distribution of travel by purpose and trip-length groupings and travel on the different highway systems by rural and urban residents is also useful to research and trade associations. These data are used by industry and marketing researc staffs of the automotive, fuel, and tire manu facturing groups for estimating the needs fo highway services.

Summary

Some of the principal findings from con sideration of data obtained in the Sta studies and Census survey are summarized the following paragraphs.

Distribution of automobiles

Automobile ownership by occupied dwellin unit was reported in both the State motor vehicle-use studies and the Census survey Automobiles were reported for occupants more than 80 percent of the dwelling units i unincorporated places, about 60 percent those in incorporated places having a popula tion of 100,000 or more, and 75 percent those in incorporated places having popula tions of less than 25,000. The relation betwee automobile ownership and family income wa a feature of the Census survey conducted i the spring of 1961. Of the families havin an annual income of more than $5,000, 9 percent or more reported owning automobiles When the annual income was less than $2,00 fewer than 40 percent of the families ha automobiles.

Motor-vehicle operators

According to the State motor-vehicle-use studies, about four-fifths of all males and two-fifths of all females of driving age were licensed operators, and more than two-thirds of all persons between the ages of 21 and 50 were licensed operators. About 90 percent of all males and slightly more than 53 percent of all females between the ages of 21 and 39 were licensed operators, and 68 percent of all males and 20 percent of all females between the ages of 60 and 69 were licensed operators.

Travel to and from work

The automobile was the principal means of transportation to and from work according to data collected in the State studies. Two out of every three workers traveled to work in automobiles and one in seven workers was a passenger in an automobile. Fifteen percent of all workers used public transportation and 12 percent walked to work. Among different occupational groups, 40 percent of the personal service workers and almost 80 percent of the craftsmen and skilled laborers traveled to work by automobile. One-half of all workers living less than 1 mile from their place of employment walked to work. Eleven percent of those living from 1 to 2 miles from work also walked. More than 70 percent of the workers living from 1 to 5 miles from work traveled by private automobile.

Use characteristics for passenger cars

The number of trips and vehicle-miles of travel reported in the State motor-vehicle-use studies are discussed in the following paragraphs.

Forty-six percent of all trips and 41 percent of all miles traveled in passenger cars were related to earning a living. For residents of incorporated places, a higher proportion of passenger-car trips and total miles of travel was for trips related to earning a living than the proportion for residents in unincorporated areas.

Trips for family business accounted for 29 percent of all trips and 19 percent of all travel; 18 percent of all trips and 34 percent of all travel was for social and recreational purposes. The average length of one-way trips for all purposes was 8 miles; the range in length among the different purposes was from 4.1 miles for educational, civic, and religious purposes to 6.4 miles for work trips and to 296 miles for vacations.

The average occupancy rate for all trips was 1.7 persons per trip. Average occupancy per trip for different purposes was: 1.3 for trips related to earning a living, 1.9 for trips for family business, and 2.4 for trips for educational, civic, religious, social, and recreational activities. Principal operators of the automobiles were responsible for 88 percent of all vehicle-miles traveled and 86 percent of all trips. Housewives drove for 10 percent of all reported passenger-car vehicle-miles.

The following information was obtained from the Census survey conducted in the spring of 1961. The average trip length for all days of the week was 8 miles; the range in length among the days of the week was from 7 miles on Thursday to 10.6 miles on Sunday. The largest proportion of the trips and travel related to earning a living were made by automobile on Mondays and Fridays. The largest proportion of trips for shopping were made on Saturdays. Almost half of all trips and three-fifths of all travel for social and recreational purposes was accomplished on weekends.

Description and Status of Studies

Motor-vehicle-use studies are designed to obtain information about the characteristics of ownership and use of motor vehicles and were first conducted in most States during the years 1935 to 1940. As the number of vehicles manufactured has increased and their characteristics have changed substantially since the earlier studies, more nearly current information has been needed for research and planning purposes. State and national automobile-use studies were conducted to obtain this needed data.

State motor-vehicle-use studies

The State motor-vehicle-use studies are conducted by the State highway departments as projects under the highway planning program and have the technical cooperation and financial assistance of the Bureau of Public Roads. The basic purpose of these studies is to assemble more detailed information about the characteristics of ownership and use of motor vehicles than can be obtained from the regular traffic count surveys. The types of information collected include:

. Ownership of automobiles and/or trucks related to population size of the owners' places of residence.

. Distribution of motor-vehicle operators according to age and sex.

. Mode of transportation used for home-to-work travel related to distance, occupation, and population size of the automobile owners' places of residence.

. Travel by type of highway system related to population size of the automobile owners' places of residence.

. Trips made and miles traveled in passenger cars according to purpose of the individual trip and population size of the automobile owners' places of residence.

. Average length of passenger-car trips according to purpose of individual trips.

. Automobile ownership by year model and population size of the owners' places of residence.

. Estimated travel made in passenger cars according to car-year model.

. Estimated fuel consumption by passenger cars and trucks.

. Annual vehicle-miles of travel by trucks (visual classification) on each highway system.

Essentially, the motor-vehicle-use study was based on recognized statistical processes in which sampling techniques are used wherein selections are made on a probability basis. The data for each household were obtained by personal interviews with occupants of the sample dwelling units. The sample design included consideration of both rural and urban characteristics of the State. The procedure followed in all but the earlier studies provided for a full year coverage of ownership and travel data: the interview sample in each population group was divided into four equal segments, and a sampling was taken each season.

The interviewers were selected, trained, and supervised by State highway planning personnel. The sample units were preselected by the study supervisors and the interviewers were not permitted to make substitutions. A few State highway departments contracted with the U.S. Bureau of the Census to perform some phases of the study, including preparation of the sample design, accomplishment of interviews, and contributions of other technical assistance. The services performed by Census varied from State to State. These State studies in which Census participated were not the same as the nationwide automobile-use study, which was conducted in its entirety by the Bureau of the Census under contract to Public Roads. Since 1951, the 24 States shown in figure 1 have conducted motor-vehicle-use studies. Partial or complete reports have been received from all of them. A few States have prepared popularized versions of the reports for general distribution.

Nationwide automobile-use study

In 1959 the Bureau of Public Roads contracted with the Bureau of the Census to collect on a nationwide interview-sample basis specific information on characteristics of ownership and use of automobiles. Although almost half of the States had conducted motor-vehicle-use studies, Public Roads desired confirmation of the findings and additional data. Reliable information was needed to establish benchmark data and to determine factors for estimating future trends in automobile ownership, vehicle mileage, and revenues from motor-vehicle user taxes. This information was also urgently needed on a national and regional basis for the Highway Cost Allocation Study required by section 210 of the Federal Highway Revenue Act of 1956 (70 Stat. 387), as amended by section 2 of the Act approved August 28, 1958. Census conducted the first survey in the fall of 1959 and the second in the spring of 1961. The principal purpose of the spring 1961 survey was to measure patterns of use in the spring in comparison with patterns of use determined in the survey of fall 1959 and to obtain some socio-economic-location information related to households.

Principal items of motor-vehicle-use information collected in the Census surveys were:

. Households classified by density of automobile ownership, total family income, and number of adults in households.

. Distribution of automobile trips generated from places of each population-size group and classification of automobile trips according to length and purpose of each trip.

. Distribution of automobile vehicle-miles generated from places of each population-size group and classified as to length and purpose of each trip.

Table 1.—Distribution of automobile ownership per occupied dwelling unit, classified by location and data from three surveys [1]

Location of dwelling units	Automobile ownership per occupied dwelling unit														
	1			2			3 or more			1 or more			None [2]		
	Motor-vehicle-use	National auto-use		Motor-vehicle-use	National auto-use		Motor-vehicle-use	National auto-use		Motor-vehicle-use	National auto-use		Motor-vehicle-use	National auto-use	
		1959	1961		1959	1961		1959	1961		1959	1961		1959	1961
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Unincorporated areas	67.6	64.3	61.9	12.4	15.6	20.4	0.9	1.8	2.3	80.9	81.7	84.6	19.1	18.3	15.4
Incorporated places, populations:															
Less than 5,000	65.2	59.8	59.5	9.1	15.1	15.4	0.7	2.1	1.7	75.0	77.0	76.6	25.0	23.0	23.4
5,000–24,999	63.2	61.0	55.0	10.8	14.9	19.5	0.9	1.0	1.7	74.9	76.9	76.2	25.1	23.1	23.8
25,000–99,999	63.0	51.4	57.2	10.3	15.0	15.0	1.0	1.6	1.0	74.3	68.0	73.2	25.7	31.9	26.8
100,000 and more	52.9	50.2	44.9	8.6	9.6	9.2	0.7	0.6	0.8	62.2	60.4	54.9	37.8	39.6	45.1
TOTAL	61.8	57.4	56.0	10.4	13.6	16.4	0.8	1.3	1.7	73.0	72.3	74.1	27.0	27.7	25.9

[1] Motor-vehicle-use data based upon summary information from 23 State studies. Data from the national automobile-use studies based upon fall 1959 and spring 1961 studies by the Bureau of the Census for Public Roads.
[2] For motor-vehicle-use data, "none" indicates no vehicles of any kind; for national auto-use surveys, "none" refers only to automobile ownership.

Table 2.—Density of automobile ownership related to percentage of dwelling units by location [1]

Location of dwelling units	Total dwelling units	Density of automobile ownership			
		1	2	3 or more	None [2]
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Unincorporated areas	31.9	34.9	37.9	33.7	22.6
Incorporated areas, populations:					
Less than 5,000	12.5	13.1	10.9	10.6	11.5
5,000–24,999	14.6	15.0	15.2	16.5	13.7
25,000–99,999	11.9	12.1	11.8	14.6	11.4
100,000 and over	29.1	24.9	24.2	24.6	40.8

[1] State motor-vehicle-use data based upon summary information from 23 States: studies were conducted between 1951 and 1958.
[2] No automobiles or trucks.

The sample used by Census for these studies was one CPS (Current Population Survey) rotating panel, or approximately 4,000 dwelling units. The CPS is conducted each month by the Bureau of the Census with a scientifically selected sample representing the noninstitutional civilian population. The main purpose of the survey is to obtain current information on employment, unemployment, and related data, which are compiled monthly. During a designated week in each month, interviewers visit the sample households and obtain the needed information.

For the national automobile-use survey, the Census interviewers obtained the needed information on automobile use from a sample of the households included in the regular CPS panel. The selected households were surveyed by mail or by personal interview and were asked to keep a travel log for 3 assigned days on each automobile owned by members of the household. This travel log provided space for recording separately each trip made, the major purpose of the trip, and the mileage. When the interviewer picked up the completed travel logs, he also filled out an interview form for each household. The interview form was used to record information such as mode of transportation to work, distance to nearest public transportation to work, and distance to nearest public transportation to main business district of the town. As the households surveyed for the automobile-use study had previously been surveyed to obtain other economic data, Census provided information on such economic items as income and composition of families, which is not available from the standard State motor-vehicle-use studies.

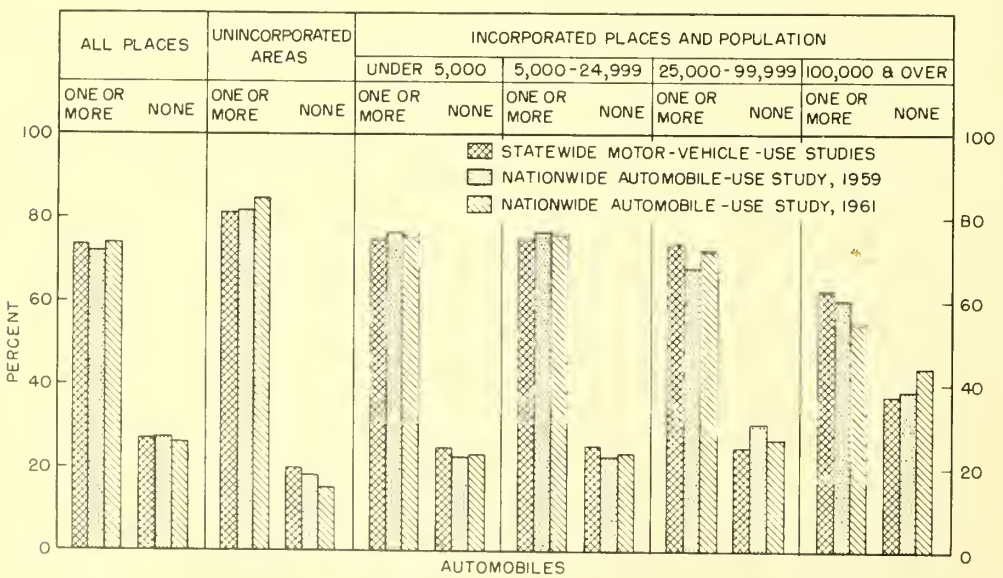

Figure 2.—Distribution of automobile ownership based on motor-vehicle-use studies and national automobile-use surveys.

Characteristics of Ownership of Motor Vehicles

Distribution of automobiles

The relative density of automobile ownership related to occupied dwelling units was reported in both the State motor-vehicle-use studies and the two national surveys conducted for Public Roads by the Bureau of the Census. Table 1 and figure 2 show a comparison between the results of these surveys. Between the years of the earlier motor vehicle-use studies, 1951–58, and the national studies conducted in the fall of 1959 and the spring of 1961, ownership patterns may have changed appreciably. Although the percentage of all occupied dwelling units for which occupants had one or more automobiles was substantially the same in the three sets of data, ranging from 72.3 to 74.1 percent, the evidence of increasing multiple ownership of automobiles can be perceived. Percentage distribution of passenger cars in relation to occupied dwelling units was highest in the unincorporated areas and decreased steadily as the size of the incorporated places increased

Dwelling units whose occupants had only one automobile comprised 61.8 percent of the total according to the motor-vehicle-use studies, but only 57.4 and 56.0 percent, respectively, as reported in the 1959 and 1961 national automobile-use surveys. Similar information collected by the Bureau of the Census during census week in 1960 (*9*) indicated that in 57.0 percent of the occupied dwelling units, ownership was reported for only one automobile.

The percentage of the total occupied dwelling units for which occupants reported ownership of two or more automobiles suggests a possible growth pattern. The motor-vehicle-use studies conducted between 1951 and 1958 showed that for 11.2 percent of the dwelling units occupants had two or more automobiles, and the 1959 and 1961 surveys showed occupants of 14.9 and 18.1 percent, respectively, of the dwellings as owning two or more. Information collected during census week in 1960 (*9*) indicated that occupants of 21 percent of the dwelling units had two or more automobiles. Multiple-car ownership was highest per dwelling unit for those located in the unincorporated areas. In general, the larger the population of the incorporated place, the percentage of dwelling units for which ownership of more than one automobile was reported decreased.

Table 2 shows the percentage distribution of automobile ownership per dwelling unit in each population-size group. The data shown in this table substantiate the assumption of greater density of automobile ownership for persons living in unincorporated areas and incorporated places having populations of less than 100,000 than for those in places where the population is greater. For example, 32 percent of all dwelling units were located in the unincorporated areas where only 23 percent of the households reported no automobiles. But for the 29 percent of the dwelling units in places having a population of more than 100,000, 41 percent of the households reported no automobiles.

Distribution by occupational groups

Table 3 and figure 3 show the distribution of automobiles, trips, and travel by the occupational-group classification of the head of the household. This information is based on the findings of the national automobile-use studies.

Table 3.—Percentage distribution of trips, travel, vehicle-miles, and automobiles for labor force, classified in groups, according to occupation of head of household

Occupation of head of household	Distribution of—						
	Persons[1] in labor force, 1959	Automobiles [2]		Trips [2]		Travel [2]	
		1959	1961	1959	1961	1959	1961
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Professional, technical, and kindred workers	10.9	11.8	16.6	13.2	19.4	11.6	19.3
Farmers and farm managers	4.6	7.7	5.7	7.1	5.1	8.1	5.8
Managers (except farm), officials and proprietors	10.6	15.6	17.3	16.8	16.4	16.1	16.5
Craftsmen, foremen, and kindred workers	13.1	21.0	20.0	20.7	20.4	21.1	19.3
Subtotal	39.2	56.1	59.6	57.8	61.3	56.9	60.9
Clerical and kindred sales workers	20.9	14.5	13.3	14.4	13.1	15.4	14.4
Operatives and kindred workers, laborers, and farm laborers	27.6	24.6	21.1	23.8	20.4	24.4	19.1
Private household and service workers	12.3	4.8	6.0	4.0	5.2	3.3	5.6
Subtotal	60.8	43.9	40.4	42.2	38.7	43.1	39.1
TOTAL	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] Employment and Earnings, Department of Labor, Bureau of Labor Statistics.
[2] National automobile-use studies fall of 1959 and spring of 1961; trips and vehicle-miles reported by heads of households who were not in labor force, members of the Armed Forces, and did not report their occupations have been omitted.

As shown in table 3, persons listed in the first four occupational groups—(1) professional, technical, and kindred workers; (2) farmers and farm managers; (3) other managers, officials, and proprietors; and (4) craftsmen, foremen, and kindred workers—proportionately performed more trips and travel and owned more vehicles than other groups of workers in the labor force. These four occupational groups represented 39 percent of the labor force and performed from 57 to 60 percent of the total travel. The other groups of workers—(1) clerical and sales workers, (2) operatives and laborers, and (3) household and service workers—owned proportionally fewer vehicles, made fewer trips, and traveled less. The relatively low ownership of vehicles by household and service workers probably is the result of their economic status.

Distribution by family income groups

Table 4 shows the distribution of automobile ownership by family income groups. This information was derived from the national automobile-use survey conducted in the spring of 1961. These figures clearly demonstrate that as the income goes up, the density of automobile ownership increases. As the yearly family income rises to more than $2,000, the percentage of the income group having automobiles increases sharply. The percentage of families in the income groups having automobiles rose steadily, reaching 98.4 percent for the group having an income of $15,000 and more. The families reporting ownership of two automobiles were concentrated in the family income groups of $6,000 and more and the families having three or more automobiles were concentrated in the family income groups of $10,000 and more.

Although income is an important consideration in automobile ownership, in many of the larger cities such factors as the lack of parking facilities and adequate public transportation systems also affect automobile ownership. And families living and working in rural areas, where there is no adequate public transportation system, must depend upon the automobile for work, shopping, and other necessary purposes even though the individual family income may be low. Table 5 shows data useful for a comparison by States between the percentage of occupied dwelling units for which ownership of vehicles, automobiles and/or trucks was reported, and the per capita personal income rank in 1960. Vehicle ownership data in this table were compiled from State motor-vehicle-use studies.

Reports from five States (Tennessee, Kentucky, Arkansas, Mississippi, and Louisiana) showed that occupants of less than 70 percent of their dwelling units had vehicles; also,

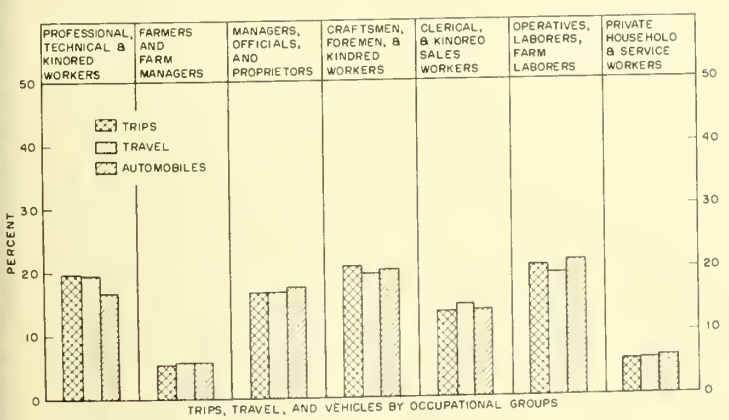

Figure 3.—Distribution of trips, travel, and vehicles classified by occupation of head of household, 1961.

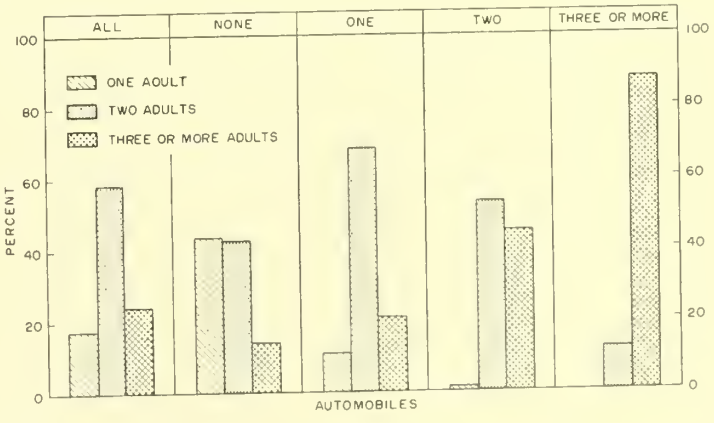

Figure 4.—Distribution of automobile ownership by number of adults in household.

Table 4.—Distribution of automobile ownership [1]

Total family income	Automobile ownership			
	1	2	3 or more	None
DISTRIBUTION OF OWNERSHIP BY FAMILIES				
Income groups	*Percent*	*Percent*	*Percent*	*Percent*
Under $1,000	8.3	4.2	1.3	1.2	21.9
$1,000–$1,999	8.7	5.2	1.5	--------	21.5
$2,000–$2,999	8.4	8.1	3.2	--------	13.0
$3,000–$3,999	8.8	9.3	4.9	4.1	10.4
$4,000–$4,999	8.6	10.1	5.2	5.4	7.7
$5,000–$5,999	10.3	14.1	7.3	6.7	4.1
$6,000–$7,499	13.8	17.0	17.9	11.6	4.5
$7,500–$9,999	9.3	10.5	15.9	13.6	2.3
$10,000–$14,999	9.3	8.9	22.0	25.1	1.3
$15,000 and more	2.6	1.7	7.7	21.3	0.2
Income not reported	11.9	10.9	13.1	11.0	13.1
TOTAL	100.0	100.0	100.0	100.0	100.0
DISTRIBUTION OF OWNERSHIP WITHIN INCOME GROUP					
Under $1,000	100.0	28.4	2.7	0.2	68.7
$1,000–$1,999	100.0	33.3	2.8	--------	63.9
$2,000–$2,999	100.0	54.0	6.1	--------	39.9
$3,000–$3,999	100.0	59.4	9.2	0.8	30.6
$4,000–$4,999	100.0	65.9	9.9	1.0	23.2
$5,000–$5,999	100.0	76.9	11.6	1.1	10.4
$6,000–$7,499	100.0	68.9	21.3	1.4	8.4
$7,500–$9,999	100.0	63.1	28.1	2.4	6.4
$10,000–$14,999	100.0	53.2	38.7	4.5	3.6
$15,000 and more	100.0	36.3	48.5	13.6	1.6
Income not reported	100.0	51.6	18.2	1.5	28.7
TOTAL	100.0	56.0	16.4	1.7	25.9

[1] National automobile-use study, spring 1961.

Table 5.—Percentage of dwelling units whose occupants owned vehicles, per capita personal income rank, and year of motor-vehicle-use study

State	Percent of dwelling units whose occupants owned vehicles [1]	Per capita [2] personal income rank 1960	Year
North Dakota	96.0	39	1951
Wyoming	89.9	18	1952–53
Idaho	89.1	40	1957–58
South Dakota	88.8	34	1952–53
Oregon	85.7	17	1953–54
Kansas	84.2	27	1957
Colorado	81.7	15	1955–56
New Mexico	81.3	41	1954
Wisconsin	81.1	23	1951
Montana	81.0	28	1952–53
Iowa	80.3	29	1951–52
Washington	79.3	14	1952–53
Hawaii	77.2	22	1952
California	76.9	6	1953
Oklahoma	75.8	38	1951
Missouri	73.8	20	1951–52
Illinois	72.9	9	1957–58
Pennsylvania	71.3	16	1952–53
Delaware	70.1	1	1953
Tennessee	69.0	46	1954
Kentucky	67.9	47	1953–54
Arkansas	62.8	50	1951
Mississippi	55.7	51	1952–53
Louisiana	55.1	43	1951

[1] Motor-vehicle-use studies include automobiles and/or trucks.
[2] Data from National Industrial Conference Board. Income ranged from a low of about $1,200 in Mississippi to a high of almost $3,100 per capita in Delaware. U.S. average was $2,242.

average per capita personal income in these States ranked relatively low. This would seem to infer that income influenced vehicle ownership. However, the three States of North Dakota, South Dakota, and Idaho reported relatively low per capita personal income but a high proportion of vehicle ownership in relation to number of dwelling units. Thus, per capita personal income may not be the only factor influencing car ownership. However, the results shown in table 5 may have been affected by the fact that the State motor-vehicle-use studies were conducted between 1951 and 1958 but the per capita personal income rank shown was for 1960. Between 1951 and 1960 the per capita personal income rank for a particular State may have changed appreciably, particularly in the agricultural States.

Distribution by number of adults in households

The number of persons old enough to operate motor vehicles (persons 16 years of age or older) in a household tends to help set the pattern for the number of automobiles owned in a household. Table 6 and figure 4 show for each automobile ownership class the number of these potential operators in a household. A high proportion, 54 percent, of the households for which ownership of two automobiles was reported were the two-adult

Table 6.—Distribution of automobile ownership by number of adults in the household [1]

Number of automobiles owned	Distribution of sample of households	Number of adults in household [2]			
		1	2	3 or more	Unknown
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
None	25.9	43.5	42.6	13.9	-----
1	56.0	10.8	68.2	20.9	0.1
2	16.4	1.1	53.6	45.3	-----
3 or more	1.7	----	12.1	87.9	-----
TOTAL	100.0	17.5	58.3	24.2	-----

[1] Nationwide automobile-use survey, spring 1961.
[2] 16 years of age or older.

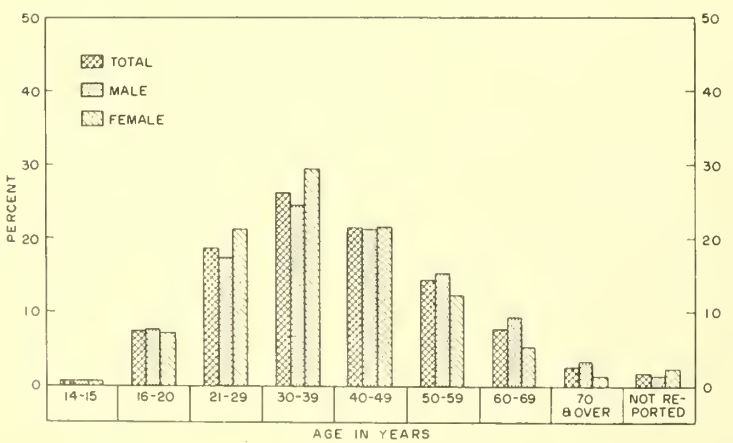

Figure 5.—Distribution of licensed motor-vehicle operators by age group and sex.

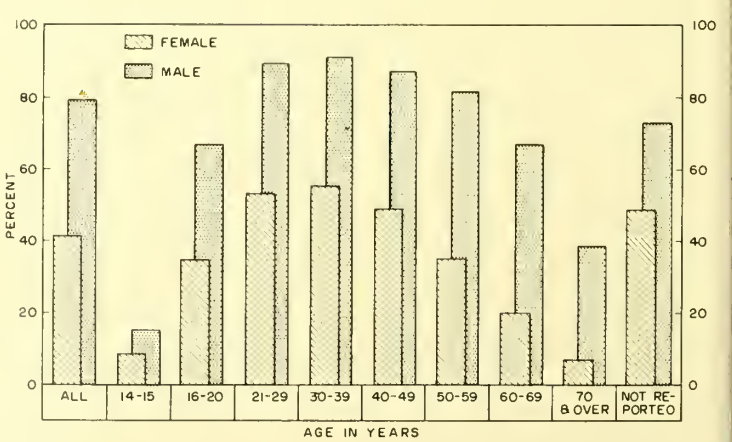

Figure 6.—Proportion of population in each age and sex group licensed as motor-vehicle operators.

Table 7.—Age distribution of licensed motor-vehicle operators and the proportion of the total population of operators in each age group [1]

Age	Distribution of licensed operators			Proportion of total population licensed as operators		
	Male	Female	Total	Male	Female	Total
Years	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
14–15	0.3	0.3	0.3	15.1	8.1	11.7
16–20	7.5	7.0	7.3	66.8	34.3	50.4
21–29	17.3	21.3	18.7	89.2	52.5	69.6
30–39	24.4	29.3	26.1	91.2	55.6	72.5
40–49	21.1	21.3	21.2	87.6	48.4	67.8
50–59	15.5	12.2	14.3	81.8	34.8	57.9
60–69	9.3	5.2	7.9	67.5	20.0	43.1
70 and over	3.3	1.1	2.5	39.4	7.1	22.5
Not reported	1.3	2.3	1.7	73.1	48.4	58.4
ALL	100.0	100.0	100.0	79.1	41.1	59.4

[1] Motor-vehicle-use studies conducted in 22 States: Arkansas, California, Colorado, Idaho, Illinois, Iowa, Kansas, Kentucky, Louisiana, Mississippi, Missouri, Montana, New Mexico, North Dakota, Oklahoma, Oregon, Pennsylvania, South Dakota, Tennessee, Washington, Wisconsin, and Wyoming.

household. For households where ownership of three or more automobiles was reported, the modal household had three or more adults.

Motor-Vehicle Operators

Information about motor-vehicle operators is important to Federal, State, and local officials in the highway taxation and planning fields. Forecasts must be made of the number of drivers who may be operating vehicles on the Nation's highways in 5, 10, or even 25 years. The proportion of drivers to the total population in the driving-age groups is expected to increase appreciably in the next few years. This increase probably will be accompanied by a proportional increase in the number of automobiles being driven for the different purposes.

In 1961 (*6*) it was estimated that almost 89 million persons were licensed to operate motor vehicles. This represented an average of 1.2 operators' permits for each registered motor vehicle; the range for licensed operators among the different States was from 105,000 in Alaska and 175,000 in Nevada to 7 million in New York and 9 million in California. The ratio of the number of licenses issued to the number of vehicles registered ranged from less than one operator per vehicle in Montana and Nevada (more vehicles registered than licenses in force) to 1.3 operators per vehicle in Massachusetts. By 1966, 131 million persons are expected to be eligible to drive, according to age requirements (*5*, p. 264). Even if the proportion of drivers to the total population remains constant, although it is expected to increase, there will be about 91 million licensed motor-vehicle operators in 1966.

Table 7, which is based on the State motor-vehicle-use studies, shows the distribution of licensed motor-vehicle operators separately for males, females, and all persons, and the proportion of the total population in each age group licensed as motor-vehicle operators. These data are shown also in figures 5 and 6.

The highest proportion of licensed operators were in the 30–39 age group; 24 percent of all male drivers and 29 percent of all female drivers were in this age group. The age groups from 21–29 and 40–49 years also had a high proportion of licensed drivers. A high proportion of persons in the age groups from 21 through 49 are expected to retain their licenses to operate a vehicle as they become older. If States enact legislation that would require a periodic reexamination for drivers, some potential drivers may be lost, particularly in the higher age groups.

Table 7 also shows the proportion of the total population in each age group that is licensed to operate a motor vehicle. Again the highest proportion, 72 percent, of licensed operators is in the 30–39 age group; 91 percent of all males and 56 percent of all females in this age group are licensed operators. An interesting statistic is the relative stability of the percentage of total males licensed to drive in the four age groups from 21 to 59 years of age; 82 to 91 percent of the males in these age groups are licensed operators. For females the stability carries only from ages 21 through 49, where from 48 to 56 percent of the females are licensed drivers. Only 20 percent of the females in the 60–69 age group and 7 percent of the females over 70 years of age are licensed operators. Many of the females in the older age groups probably never did learn to drive.

Travel To and From Work

Most workers today have the problem of commuting to work. Joining the rush-hour traffic to get to and from work is now an accepted part of the routine of urban living. As the population and the number of workers grow and more and more families settle in the suburban areas, the distance necessary to be traveled to work tends to be increased.

Based upon summary data developed from 21 State motor-vehicle-use studies, 80 percent of all gainfully employed workers use some form of travel to get to work. Independent data collected by the Bureau of the Census in 1960 showed that 93 percent of all workers required transportation to work. Some of the differences in these two sets of data are traceable to differences in definition of "work at home." In the motor-vehicle-use studies, a person is considered as being gainfully employed at home if he regularly conducts his business from his place of residence; this includes a doctor, plumber, or traveling salesman. In the survey conducted at 10-year intervals by the Bureau of the Census, a doctor or salesman who uses an automobile in connection with work, even though operating his business from his place of residence, would be reported as using an automobile for transportation to work. Other differences in the results of these two studies may be the result of the difference in time of survey. The 21 State motor-vehicle-use studies were conducted in different years, between 1951 and 1958. The Census study was national in scope and was conducted for a week in April 1960.

Table 8 and figure 7 show by occupational and population groupings the percentage of workers requiring transportation to get to work. For residents of all places the range was from 46 percent for proprietors, managers, and officials (including farmers and farm managers) to 96 percent for store and office clerks. Fifty-nine percent of all workers in

Table 8.—Percentage of workers in each occupational group required to travel [1] to get to work, classified by residence of operator [2]

Occupational group of operator	Residence of principal operator						
	All places	Unincorporated areas	Incorporated places having populations of—				
			Less than 5,000	5,000–24,999	25,000–99,999	100,000 and more	Total
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Professional and semiprofessional	87.9	88.0	82.4	90.8	92.4	86.5	87.8
Proprietors, managers, and officials, including farmers and farm managers	46.4	20.1	78.5	86.5	89.2	85.3	84.6
Store and office clerks, salesmen (excluding traveling salesmen)	95.9	94.4	92.1	94.6	97.4	97.6	96.3
Traveling salesmen, agents	85.1	79.0	77.0	82.8	88.6	89.7	86.3
Craftsmen, foremen, skilled laborers	94.4	93.1	91.0	94.7	96.5	96.0	95.0
Operatives, semiskilled workers, unskilled workers, laborers	89.1	77.7	91.8	94.7	96.3	97.7	95.7
Protective services workers	61.4	46.0	76.6	54.7	88.1	71.8	69.7
Personal service workers	87.3	80.8	85.7	87.1	86.7	90.6	88.7
Miscellaneous [3]	63.9	61.8	62.4	52.1	79.5	67.3	64.0
TOTAL	80.4	59.4	86.9	89.9	93.7	92.9	91.4

[1] Includes walking.
[2] Motor-vehicle-use studies conducted in 21 States: Arkansas, California, Delaware, Idaho, Illinois, Iowa, Kansas, Kentucky, Louisiana, Mississippi, Missouri, Montana, New Mexico, North Dakota, Oklahoma, Oregon, Pennsylvania, South Dakota, Tennessee, Washington, and Wyoming.
[3] Includes workers not reporting occupation.

Table 9.—Distribution of workers traveling from home to work classified by mode of travel within each occupational group—residents of all places, all unincorporated areas, and all incorporated places [1]

Residence location and occupational group	Automobile			Public transportation	Automobile and public transportation	Walk	All other means and not reported
	Driver	Passenger	Total				
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
All places:							
Professional and semiprofessional	58.8	12.6	71.4	13.0	2.4	10.6	2.6
Proprietors, managers, officials [2]	71.0	6.2	77.2	5.8	1.5	11.9	3.6
Store and office clerks, salesmen	40.4	18.1	58.5	24.9	2.4	12.4	1.8
Traveling salesmen, agents	71.3	4.6	75.9	12.8	3.7	4.8	2.8
Craftsmen, foremen, skilled laborers	65.7	13.5	79.2	10.1	1.3	7.0	2.4
Operatives, workers, and laborers	49.2	19.2	68.4	14.2	1.0	13.6	2.8
Protective services workers	63.3	11.7	75.0	10.1	1.5	6.4	7.0
Personal service workers	25.2	14.8	40.0	31.3	2.0	24.3	2.4
Miscellaneous [3]	33.3	12.0	45.3	12.1	2.1	9.5	31.0
ALL OCCUPATIONS	53.7	14.8	68.5	15.1	1.7	11.8	2.9
Unincorporated areas:							
Professional and semiprofessional	67.6	15.5	83.1	6.8	1.6	5.0	3.5
Proprietors, managers, officials [2]	72.0	6.0	78.0	2.0	1.2	13.7	5.1
Store and office clerks, salesmen	54.2	25.1	79.3	11.0	2.5	5.2	2.0
Traveling salesmen, agents	80.8	7.4	88.2	4.4	3.5	2.5	1.4
Craftsmen, foremen, skilled laborers	74.1	15.7	89.8	3.0	1.3	3.4	2.5
Operatives, workers, and laborers	60.0	23.4	83.4	3.7	0.9	8.4	3.6
Protective services workers	68.4	11.6	80.0	4.4	0.8	6.9	7.9
Personal service workers	44.1	26.3	70.4	10.2	1.4	14.2	3.8
Miscellaneous [3]	38.5	12.3	50.8	10.2	6.0	4.9	28.1
ALL OCCUPATIONS	64.2	18.6	82.8	4.9	1.4	7.3	3.6
All incorporated places:							
Professional and semiprofessional	56.6	11.8	68.4	14.6	2.5	12.1	2.4
Proprietors, managers, officials [2]	70.6	6.4	77.0	7.2	1.6	11.3	2.9
Store and office clerks, salesmen	37.4	16.6	54.0	27.9	2.4	14.0	1.7
Traveling salesmen, agents	69.6	4.1	73.7	14.3	3.8	5.2	3.0
Craftsmen, foremen, skilled laborers	62.3	12.7	75.0	13.0	1.3	8.4	2.3
Operatives, workers, and laborers	44.1	17.3	61.4	19.1	1.0	16.0	2.5
Protective services workers	61.6	11.8	73.4	12.0	1.7	6.2	6.7
Personal service workers	21.5	12.6	34.1	35.3	2.2	26.2	2.2
Miscellaneous [3]	31.3	11.9	43.2	12.9	0.5	11.3	32.1
ALL OCCUPATIONS	50.1	13.5	63.6	18.6	1.8	13.4	2.6

[1] 21 State motor-vehicle-use studies: Arkansas, California, Delaware, Idaho, Illinois, Iowa, Kansas, Kentucky, Louisiana, Mississippi, Missouri, Montana, New Mexico, North Dakota, Oklahoma, Oregon, Pennsylvania, South Dakota, Tennessee, Washington, and Wyoming.
[2] Includes farmers and farm managers who traveled to work.
[3] Includes workers not reporting occupation.

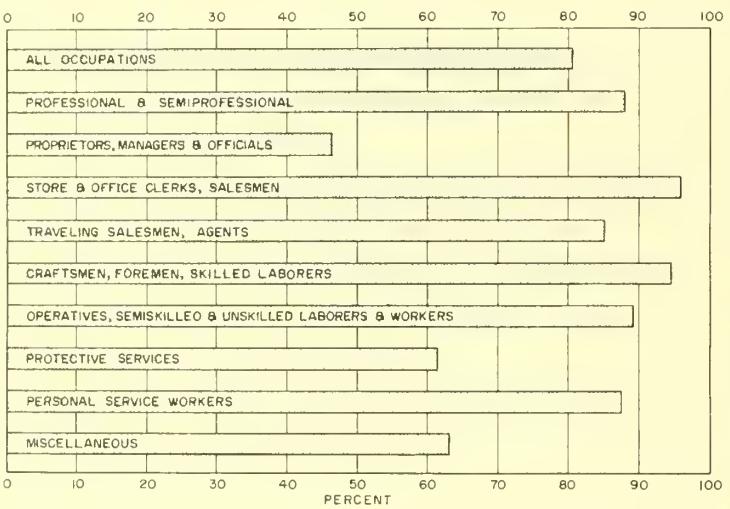

Figure 7.—Percentage of workers in each occupational group that requires travel to reach place of employment.

the unincorporated areas traveled to work as compared with 91 percent in the incorporated places.

Modes of travel to and from work

According to the motor-vehicle-use studies, 68 percent of all workers travel to and from work by automobile, either as the driver or as a passenger. An additional 15 percent use public transportation. Table 9 indicates the extent of use of different modes of transportation by each occupational group for residents of all places. In addition, table 9 shows comparable data separately for residents of unincorporated areas and incorporated places. For residents of all places, workers using automobiles for home-to-work travel ranged from 40 percent for personal service workers to 79 percent for craftsmen, foremen, and skilled laborers. Almost one-third of all the personal service workers used public transportation to get to work.

A relatively high proportion of store and office clerks, 25 percent, also used public transportation to get to and from work. However, less than 6 percent of the occupational group, including proprietors, managers, and officials, went to work by public transportation. About 12 percent of all workers walked to work. The number of walkers among the principal occupational groups ranged from 6 percent for workers in the protective services to 24 percent for the personal service workers.

Workers using a combination of methods to get to work, that is automobile and public transportation or some other combination, accounted for a relatively small percent of the total modes of travel.

Relation of Distance to Work and Mode of Travel

The mode of travel to work is influenced by such factors as distance, type and convenience of public transportation, and occupation. Usually the workers who live in the rural areas have less public transportation available. Table 10 shows the mode of travel used by workers grouped according to distance.

Walking was the most popular method of getting to work where the individuals lived less than 1 mile from work; more than half of all workers in that mileage group walked to work and 43 percent went by private automobile. Where the distance to work was more than 2 miles, few walked. Workers using a combination of passenger cars and public transportation to get to work are included mainly in the 13-miles-and-more distance groups. The highest percentage, 6.6 percent, of workers using a combination method of transportation lived 25 miles or more from their work.

Public transportation was the most popular method of travel for those living 2 to 9 miles from work; 20 to 25 percent of the workers in this distance group used public transportation. As people live farther from work, particularly when the distance is more than 15 miles, public transportation is used less possibly because of inconvenient scheduling of public transportation.

A high proportion of automobile drivers or riders is shown in all mileage groupings. Except for those workers living less than 1 mile from work, more than 70 percent of the workers went to work by private automobile. Also, more than 80 percent of the workers living more than 11 miles from work either rode or drove in automobiles. Only 7.8 percent of persons living less than 1 mile from work were passengers in private automobiles. Where the distance was from 1 to 14 miles, from 15 to 17 percent of persons were reported as being automobile passengers. More than 20 percent of the persons living 14 miles and more from work were passengers in private automobiles. This may reflect the influence of car pooling from the more distant

points and the lack and/or inadequacy of public transportation.

Use Characteristics for Automobiles

Travel to and from work accounts for more than one-third of all automobile trips and more than one-fourth of all passenger-car travel, according to the motor-vehicle-use studies. Planners of urban highway facilities know only too well that such travel is responsible for the morning and evening rush-hour traffic peaks and accompanying congestion that provide their most knotty problems. However, the other two-thirds of the automobile trips and three-fourths of travel require attention too, especially in connection with the planning of interstate and intercity facilities, and rural feeder and access roads.

Purpose of travel

As table 11 and figure 8 data indicate, more than 46 percent of all automobile trips and almost 44 percent of all travel is related to earning a living. The proportion of total trips and travel made that is related to earning a living by residents of incorporated places is somewhat higher than the proportions for persons residing in unincorporated areas. For residents of all places, passenger cars are used for commuting to and from work on one-third of all one-way trips and more than one-fourth of all travel. Persons living in the unincorporated areas perform proportionally fewer work trips than persons living in cities having a population of 100,000 and more. Persons living in all other incorporated places travel about the same proportion of their total miles for work trips as do residents of unincorporated areas.

Trips made for purposes of family business accounted for nearly 29 percent of all trips and 19 percent of all travel. Persons living in the unincorporated areas accounted for a higher proportion of their trips and travel for such purposes than residents of incorporated places. Residents of incorporated places are usually nearer to shopping and medical service

Table 10.—Distribution of workers classified by mode of travel to work, according to distance to work[1]

Distance to place of employment	Automobile			Public transportation	Automobile and public transportation	Walk	All other means and not reported
	Driver	Passenger	Total				
Miles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
0.1–0.9	35.0	7.8	42.8	3.1	0.4	52.4	1.3
1.0–1.9	55.4	16.7	72.1	14.9	0.9	11.1	1.0
2.0–2.9	55.8	16.3	72.1	23.4	1.2	1.9	1.4
3.0–3.9	57.0	14.5	71.5	25.8	1.0	0.5	1.2
4.0–4.9	57.0	15.8	72.8	24.6	1.2	0.2	1.2
5.0–5.9	59.7	15.4	75.1	21.9	1.3	0.2	1.5
6.0–6.9	60.2	15.4	75.6	21.6	1.7	--	1.1
7.0–7.9	62.0	15.8	77.8	18.6	1.9	--	1.7
8.0–8.9	59.9	15.6	75.5	21.1	2.4	--	1.0
9.0–9.9	65.3	17.9	83.2	13.1	2.1	--	1.6
10.0–10.9	61.9	16.9	78.8	17.1	2.6	--	1.5
11.0–11.9	65.8	17.1	82.9	13.9	2.2	--	1.0
12.0–12.9	65.9	17.2	83.1	14.0	2.0	--	0.9
13.0–13.9	67.3	17.2	84.5	10.1	4.2	--	1.2
14.0–14.9	60.6	20.3	80.9	13.4	4.2	--	1.5
15.0–19.9	65.3	20.4	85.7	9.1	3.4	--	1.8
20.0–24.9	64.3	21.1	85.4	7.0	5.9	--	1.7
25.0 and more	61.8	22.2	84.0	7.5	6.6	--	1.9
Not reported	36.4	9.1	45.5	7.4	1.0	17.6	28.5
ALL DISTANCES	53.7	14.8	68.5	15.1	1.7	11.8	2.9

[1] Motor-vehicle-use studies conducted in 21 States: Arkansas, California, Delaware, Idaho, Illinois, Iowa, Kansas, Kentucky, Lousiana, Mississippi, Missouri, Montana, New Mexico, North Dakota, Oklahoma, Oregon, Pennsylvania, South Dakota, Tennessee, Washington, and Wyoming.

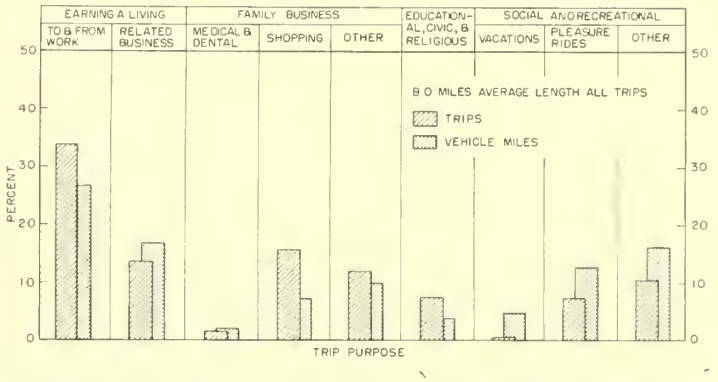

Figure 8.—Automobile trips and travel classified by major purpose of trip.

Table 11.—Distribution of automobile trips and travel by purpose and population groups[1]

Purpose of trip[2]	Residence of principal operator													
	All population groups		Unincorporated areas		Incorporated places and populations									
					Under 5,000		5,000–24,999		25,000–99,999		100,000 and more		Total incorporated	
	Trips	Travel	Trips	Travel	Trips	Travel	Trips	Travel	Trips	Travel	Trips	Travel	Trips	Travel
Earning a living:	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
To and from work	33.3	26.8	27.6	25.1	33.4	24.2	34.1	25.1	35.2	24.3	39.0	33.4	35.9	27.8
Related business	13.2	16.8	15.2	16.2	14.3	19.3	11.6	18.7	11.2	18.3	12.4	14.5	12.3	17.2
Subtotal	46.5	43.6	42.8	41.3	47.7	43.5	45.7	43.8	46.4	42.6	51.4	47.9	48.2	45.0
Family business:														
Medical and dental	1.6	1.9	1.8	2.7	1.6	2.5	1.4	1.3	1.3	1.1	1.6	1.1	1.5	1.5
Shopping	15.4	7.2	17.4	10.4	14.6	6.7	15.7	5.4	15.1	5.5	13.3	4.7	14.5	5.4
Other	11.6	9.9	11.8	10.2	10.9	8.8	11.5	10.2	12.7	9.8	11.0	9.7	11.5	9.6
Subtotal	28.6	19.0	31.0	23.3	27.1	18.0	28.6	16.9	29.1	16.4	25.9	15.5	27.5	16.5
Educational, civic, and religious	7.2	3.7	8.1	4.9	6.5	3.1	7.0	2.7	7.5	3.0	6.3	3.0	6.8	2.9
Social and recreational:														
Vacations	0.1	4.9	0.1	3.3	0.1	4.2	0.2	5.0	0.2	8.5	0.1	6.0	0.1	5.9
Pleasure rides	7.1	12.7	6.9	11.6	9.1	15.5	7.7	13.4	7.7	15.9	5.8	10.8	7.3	13.4
Other	10.5	16.1	11.1	15.6	9.5	15.7	10.8	18.2	9.1	13.6	10.5	16.8	10.1	16.3
Subtotal	17.7	33.7	18.1	30.5	18.7	35.4	18.7	36.6	17.0	38.0	16.4	33.6	17.5	35.6
ALL PURPOSES	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] Motor-vehicle-use studies conducted in 22 States: Arkansas, California, Colorado, Idaho, Illinois, Iowa, Kansas, Kentucky, Louisiana, Mississippi, Missouri, Montana, New Mexico, North Dakota, Oklahoma, Oregon, Pennsylvania, South Dakota, Tennessee, Washington, Wisconsin, and Wyoming.
[2] A trip is defined as a one-way movement from a starting place to the first stop for one of the purposes shown.

Table 12.—Average length of one-way trips by major purpose of trip in selected States[1]

State	All purposes	Earning a living		Family business			Educational, civic, and religious	Social and recreational		
		To and from work	Related business	Medical and dental	Shopping	Other		Vacations	Pleasure rides	Other
	Miles	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*
Arkansas	8.7	4.9	13.3	8.5	3.6	9.1	5.7	156.6	15.3	11.9
California	7.9	7.4	8.5	8.1	3.5	6.7	4.3	205.4	21.8	11.8
Colorado	7.9	5.7	7.0	5.7	3.5	6.5	3.7	269.8	18.6	12.1
Idaho	8.3	4.9	9.6	9.9	5.8	7.6	4.2	80.9	31.6	14.9
Illinois	7.0	6.5	9.5	7.1	3.4	7.0	3.1	747.9	12.0	9.2
Iowa	8.2	3.6	11.8	15.5	4.6	6.8	4.7	535.4	13.7	----
Kansas	6.2	4.6	6.2	8.0	3.6	6.1	4.3	94.1	11.1	11.8
Kentucky	10.2	6.4	15.5	10.3	4.1	7.8	5.3	654.4	12.5	15.7
Louisiana	9.3	7.6	11.9	15.4	8.8	8.8	4.5	171.4	11.9	12.3
Mississippi	8.6	6.1	14.0	13.1	4.8	6.8	3.9	408.2	15.3	11.2
Missouri	8.2	5.9	9.5	9.3	4.0	4.4	4.2	641.5	22.9	14.9
Montana	9.3	3.8	17.2	21.0	4.7	10.1	6.2	472.5	16.1	12.6
New Mexico	8.2	4.4	12.6	20.5	3.6	4.7	2.9	377.7	11.1	3.5
North Dakota	12.9	5.3	16.1	34.8	6.3	8.3	9.3	538.9	12.9	16.4
Oklahoma	8.3	6.1	9.8	11.9	3.7	----	7.1	265.9	13.4	16.5
Oregon	8.7	6.9	10.0	15.5	4.2	6.7	4.4	320.2	14.7	6.9
Pennsylvania	9.0	6.9	14.2	9.1	3.9	7.5	3.8	111.1	16.2	14.4
South Dakota	10.2	3.4	13.8	25.9	7.1	7.1	5.4	1,031.2	14.0	16.1
Tennessee	8.1	7.1	8.6	8.1	3.6	13.5	5.2	287.5	12.6	11.1
Washington	8.9	6.7	14.8	11.3	4.1	6.7	3.9	106.8	13.7	3.9
Wisconsin	7.9	5.3	9.3	23.4	3.8	6.3	3.7	264.3	15.2	11.2
Wyoming	14.7	6.4	57.0	23.6	6.5	28.4	5.5	318.9	23.6	23.3
AVERAGE	8.0	6.4	10.2	9.7	3.8	6.8	4.1	296.0	14.2	12.3

[1] Motor-vehicle-use studies conducted in each State.

areas and other places for conducting family business than persons living in the unincorporated areas. Shopping trips accounted for 15 percent of all trips but only 7 percent of all the travel. Trips made for medical and dental purposes were responsible for less than 2 percent of the trips and travel. But, trips for family business other than medical, dental, and shopping accounted for 12 percent of the trips and 10 percent of the travel. These trips included those for such purposes as to see a lawyer, insurance agent, hairdresser, or a barber.

Trips made for social and recreational purposes accounted for 18 percent of the trips and 34 percent of the travel. Persons living in either incorporated or unincorporated areas performed about the same proportion of the trips, but residents of incorporated places performed a higher proportion of their travel for social and recreational purposes than persons living in the unincorporated areas. Thirty-six percent of the travel by residents of incorporated places was for social and recreational purposes, while only 30 percent of the travel by unincorporated area residents was for such purposes. Only 0.1 percent of the trips were made for vacations but these accounted for 5 percent of the total travel.

Table 12 shows the average one-way-trip length by purpose of trips in each of 22 States. For the 22 States combined, the average one-way-trip length for all purposes was 8.0 miles, it ranged from 6.2 miles in Kansas to 14.7 miles in Wyoming. Trips to and from work were relatively short trips, the average being 6.4 miles, and ranged from 3.4 miles in South Dakota to 7.6 miles in Louisiana.

Related business trips averaged 10.2 miles for the 22 States. Nine States reported an average business trip length of less than 10 miles, nine States between 10 and 15 miles, three States between 15 and 17 miles, while Wyoming reported an average trip length of 57 miles.

Trips for shopping purposes were relatively short, averaging less than 4 miles. Similarly, trips for educational purposes averaged less than 5 miles. Vacation trips averaged almost 300 miles one way, with a wide difference in averages among the various States. One reason for the range in the averages reported among the States was the definitions used for vacations and pleasure rides. For example, in some States an overnight trip was considered a pleasure ride while in other States it was considered a vacation. Another factor probably was the availability of the major vacation areas within the State and/or adjacent States.

Comparison With Earlier Studies

Between 1935 and 1940 most State highway departments conducted road-use studies that were designed to provide more information about the characteristics of motor-vehicle use than was then available. These studies were somewhat similar to the motor-vehicle-use studies that have been conducted since 1951.

Both types of studies were based on samples. However, in the earlier studies the universe in most instances was the motor-vehicle registrations of the previous complete year. The universe for the passenger-car data in the current motor-vehicle-use studies was the total number of dwelling units in the State being studied.

The information in both groups of studies was collected through personal interview with the respondents. In a few States, the road-use data needed were obtained through the so-called "school method." In States adopting this method the study was set up as a school project in classes in selected high schools.

The interviewers employed in the more recent motor-vehicle-use studies were given no latitude in selecting persons to be interviewed. In the earlier road-use studies, quotas within areas were established and the interviewers were instructed to make their selections on a more or less random basis. Some controls were established, however, including the requirement that urban interviews be distributed according to the distribution of workers among the different occupations. In the earlier studies, an attempt was made to obtain trip information for an entire 12-month period, generally the 12-month period immediately preceding the date of the interview. The more recent studies are designed to obtain data only for trips reported on the most recent workday and Saturday and Sunday. Further, the procedures for the motor-vehicle-use studies conducted since 1952 provided that one-fourth of the interviews shall be obtained in each area sampled in each of the four seasons of the year. These changes in procedures are expected to reduce memory bias or unreliability in the information obtained and to provide uniform seasonal coverage of the areas sampled.

Despite differences in the procedures used in the two studies, it is still possible to make some significant comparisons between the two. Table 13 shows by State and population groups the percentage of total travel assignable to business and pleasure. Two sets of data are presented for each population group. The first reports the percentage of travel assignable to business and pleasure as developed from the road-use studies conducted during the late 1930's. The second set of data shows the results of the motor-vehicle-use studies conducted in the different States from 1951 through 1958.

Business travel in both studies includes travel to and from work and for related business and family business trips, such as for shopping, medical, and dental purposes. The travel shown for pleasure purposes includes the travel for social and recreational activities plus that for educational, civic, and religious purposes. Although it is difficult to draw conclusions from these two sets of data, a few general observations can be made. Most of the States shown, which are generally considered as being somewhat rural in character, reported less travel for business purposes in the motor-vehicle-use studies than in the earlier road-use studies. Also, in the major cities having a population of 100,000 or more, the more recent studies show a higher percentage of travel for business purposes than the earlier studies. This could possibly have been caused by decentralization of commercial and industrial firms. The motor-vehicle-use study results showed that people in the unincorporated areas generally drove less of their total mileage for business purposes than results reported in the earlier studies.

Automobile Occupancy

Information on automobile occupancy is essential as a base for computing passenger miles of travel in private automobiles. The data are of general interest also to highway

Table 13.—Comparison by State between distribution of travel for business and pleasure as reported in the road-use studies [1] and the motor-vehicle-use studies [2]

Purpose of travel by State	Residence of principal operator													
	All places		Unincorporated areas		Incorporated places populations									
					Less than 5,000		5,000–24,000		25,000–99,999		100,000 and more		Total	
	Road-use	Motor-vehicle-use	Road-use	Motor-vehicle-use	Road-use	Motor-vehicle-use	Road-use	Motor-vehicle-use	Road-use	Motor-vehicle-use	Road-use	Motor-vehicle-use	Road-use	Motor-vehicle-use
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Arkansas:														
Business	71.6	60.0	76.9	62.0	71.5	59.7	61.9	62.6	62.6	51.5		58.4	66.6	58.8
Pleasure	28.4	40.0	23.1	38.0	28.5	40.3	38.1	37.4	37.4	48.5		41.6	33.4	41.2
Colorado:														
Business	62.6	53.9	72.5	60.2	60.2	50.0	61.5	31.0	50.3	65.9	56.9	77.3	57.9	51.4
Pleasure	37.4	46.1	27.5	39.8	39.8	50.0	38.5	69.0	49.7	34.1	43.1	22.7	42.1	48.6
Idaho:														
Business	66.8	56.1	73.2	55.1	62.2	62.1	60.1	52.9	----	57.2			61.2	57.1
Pleasure	33.2	43.9	26.8	44.9	37.8	37.9	39.9	47.1		42.8			38.8	42.9
Illinois:														
Business	48.1	69.7	60.3	65.4	49.5	67.5	43.5	71.9	43.4	69.7	48.1	73.1	46.3	71.1
Pleasure	51.9	30.3	39.7	34.6	50.5	32.5	56.5	28.1	56.6	30.3	51.9	26.9	53.7	28.9
Iowa:														
Business	57.8	55.3	67.4	56.8	56.4	56.8	38.8	50.1	61.6	55.1	48.7	53.0	54.3	54.5
Pleasure	42.2	44.7	32.6	43.2	43.6	43.2	61.2	49.9	38.4	44.9	51.3	47.0	45.7	45.5
Kansas:														
Business	56.2	61.2	66.5	62.1	51.3	60.9	53.4	59.5	52.2	58.5	50.8	62.5	52.0	60.5
Pleasure	43.8	38.8	33.5	37.9	48.7	39.1	46.6	40.5	47.8	41.5	49.2	37.5	48.0	39.5
Kentucky:														
Business	64.6	52.5	66.2	49.7	65.4	51.8	65.9	67.3	59.8	57.7	61.3	54.0	63.1	56.5
Pleasure	35.4	47.5	33.8	50.3	34.6	48.2	34.1	32.7	40.2	42.3	38.7	46.0	36.9	43.5
Louisiana:														
Business	70.1	61.2	80.0	61.9	71.2	70.3	68.5	61.9	70.7	57.2	59.2	54.4	64.6	60.6
Pleasure	29.9	38.8	20.0	38.1	28.8	29.7	31.5	38.1	29.3	42.8	40.8	45.6	35.4	39.4
Mississippi:														
Business	67.2	67.2	70.9	73.2	66.9	61.5	64.1	67.4	61.8	59.0			65.2	62.2
Pleasure	32.8	32.8	29.1	26.8	33.1	38.5	35.9	32.6	38.2	41.0			34.8	37.8
Missouri:														
Business	63.0	62.8	73.6	71.5	67.9	61.9	59.1	63.2	65.1	52.9	57.2	58.1	60.0	59.5
Pleasure	37.0	37.2	26.4	28.5	32.1	38.1	40.9	36.8	34.9	47.1	42.8	41.9	40.0	40.5
Montana:														
Business	63.0	58.8	76.8	64.6	56.5	51.9	55.1	61.6	53.1	50.6			55.4	54.1
Pleasure	37.0	41.2	23.2	35.4	43.5	48.1	44.9	38.4	46.9	49.4			44.6	45.9
New Mexico:														
Business	55.6	61.6	55.6	69.9	54.4	67.3	56.4	55.0	55.7	55.1			55.5	57.2
Pleasure	44.4	38.4	44.4	30.1	45.6	32.7	43.6	45.0	44.3	44.9			44.5	42.8
North Dakota:														
Business	62.4	53.3	68.9	59.8	59.8	47.8	56.9	58.5	53.7	24.2			57.8	48.4
Pleasure	37.6	46.7	31.1	40.2	40.2	52.2	43.1	41.5	46.3	75.8			42.2	51.6
Oklahoma:														
Business	70.8	59.1	77.4	68.4	69.7	42.3	72.6	55.6	63.7	51.7	61.0	63.9	66.9	53.9
Pleasure	29.2	40.9	22.6	31.6	30.3	57.7	27.4	44.4	36.3	48.3	39.0	36.1	33.1	46.1
Oregon:														
Business	55.4	62.6	64.3	68.5	52.6	57.8	48.7	58.3	37.3	48.9	52.4	57.2	51.2	56.9
Pleasure	44.6	37.4	35.7	31.5	47.4	42.2	51.3	41.7	62.7	51.1	47.6	42.8	48.8	43.1
South Dakota:														
Business	56.8	57.3	65.8	56.8	54.6	63.5	51.9	47.7	48.2	62.2			53.3	57.8
Pleasure	43.2	42.7	34.2	43.2	45.4	36.5	48.1	52.3	51.8	37.8			46.7	42.2
Tennessee:														
Business	63.8	56.1	68.7	62.3	65.0	53.5	57.4	56.4	62.7	49.3	60.0	46.2	60.9	49.6
Pleasure	36.2	43.9	31.3	37.7	35.0	46.5	42.6	43.6	37.3	50.7	40.0	53.8	39.1	50.4
Washington:														
Business	62.1	63.1	67.7	65.2	62.3	54.2	58.1	63.2	56.7	63.4	60.1	62.3	59.5	61.3
Pleasure	37.9	36.9	32.3	34.8	37.7	45.8	41.9	36.8	43.3	36.6	39.9	37.7	40.5	38.7
Wisconsin:														
Business	49.6	59.3	54.5	69.6	50.2	67.7	48.8	53.1	50.7	47.5	41.9	44.1	48.2	52.8
Pleasure	50.4	40.7	45.5	30.4	49.8	32.3	51.2	46.9	49.3	52.5	58.1	55.9	51.8	47.2
Wyoming:														
Business	56.4	50.7	67.0	69.0	51.0	49.4	49.3	34.9	----	38.2	----		50.0	43.1
Pleasure	43.6	49.3	33.0	31.0	49.0	50.6	50.7	65.1	----	61.8	----		50.0	56.9

[1] Road-use studies conducted during the 1930's.
[2] Motor-vehicle-use studies conducted since 1951.

Table 14.—Average occupancy in automobile trips, classified by location of travel, major purpose of travel, by selected population groups [1]

Population group of residence of principal operator and location of travel	Occupants and major purpose of travel												
	All purposes	Earning a living			Family business				Educational, civic, and religious	Social and recreational			
		To and from work	Related business	Total	Medical and dental	Shopping	Other	Total		Vacations	Pleasure rides	Other	Total
All population groups:													
All trips	1.7	1.3	1.3	1.3	2.0	1.9	1.8	1.9	2.4	2.7	2.5	2.4	2.4
Trips partially within an incorporated place and partially rural	1.9	1.3	1.4	1.4	2.2	2.1	2.0	2.1	2.5	2.8	2.7	2.5	2.6
Trips entirely within an incorporated place [2]	1.6	1.2	1.2	1.2	1.8	1.8	1.7	1.8	2.3	2.3	2.3	2.2	2.2
Trips entirely outside incorporated places	1.9	1.3	1.4	1.4	2.0	1.9	1.9	1.9	2.7	2.9	2.6	2.5	2.5
Unincorporated areas:													
All trips	1.9	1.3	1.5	1.4	2.2	2.0	1.9	2.0	2.6	3.1	2.7	2.5	2.6
Trips partially within an incorporated place and partially rural	1.9	1.3	1.5	1.4	2.3	2.1	1.9	2.1	2.6	3.2	2.7	2.5	2.6
Trips entirely within an incorporated place [2]	1.8	1.3	1.4	1.3	2.0	1.9	2.0	2.0	2.5	3.8	2.6	2.5	2.5
Trips entirely outside incorporated places	1.9	1.3	1.5	1.4	2.0	1.9	1.8	1.9	2.7	2.6	2.6	2.5	2.5
All incorporated places:													
All trips	1.7	1.2	1.3	1.2	1.9	1.8	1.8	1.8	2.3	2.7	2.5	2.3	2.4
Trips partially within an incorporated place and partially rural	1.9	1.3	1.4	1.3	2.0	2.1	2.0	2.1	2.4	2.7	2.7	2.5	2.6
Trips entirely within an incorporated place [2]	1.6	1.2	1.2	1.2	1.8	1.8	1.7	1.8	2.3	2.2	2.3	2.1	2.2
Trips entirely outside incorporated places	1.8	1.3	1.3	1.3	2.5	1.8	2.4	2.2	2.3	3.1	2.9	2.8	2.9

[1] Motor-vehicle-use studies in 16 States: California, Colorado, Idaho, Illinois, Iowa, Kansas, Kentucky, Mississippi, Montana, New Mexico, Oregon, Pennsylvania, South Dakota, Tennessee, Washington, and Wyoming.
[2] Or within contiguous incorporated places.

Table 15.—Distribution of automobile trips by purpose of trip and location of travel [1]

Purpose of trip	Distribution by purpose	Distribution of travel		
		Trips partially within an incorporated place and partially rural	Trips entirely within an incorporated place [2]	Trips entirely outside incorporated places
Earning a living:	*Percent*	*Percent*	*Percent*	*Percent*
To and from work	33.6	36.7	56.3	7.0
Related business	12.2	42.3	46.0	11.7
TOTAL	45.8	38.2	53.6	8.2
Family business:				
Medical and dental	1.6	48.4	48.3	3.3
Shopping	15.8	30.7	58.0	11.3
Other	12.1	36.4	53.8	9.8
TOTAL	29.5	34.1	55.7	10.2
Educational, civic, and religious	7.6	28.1	57.8	14.1
Social and recreational:				
Vacations	0.1	83.9	8.3	7.8
Pleasure rides	7.2	54.6	36.1	9.3
Other	9.8	48.5	40.4	11.1
TOTAL	17.1	51.3	38.3	10.4
ALL PURPOSES	100.0	38.4	52.0	9.6

[1] Motor-vehicle-use studies conducted in 16 States: California, Colorado, Idaho, Illinois, Iowa, Kansas, Kentucky, Mississippi, Montana, New Mexico, Oregon, Pennsylvania, South Dakota, Tennessee, Washington, and Wyoming.
[2] Or within contiguous incorporated places.

Table 16.—Proportion of driving done by principal operator according to occupation and population group of residence [1]

Occupational group of principal operator	All places		Unincorporated areas		All incorporated places	
	Vehicle-miles	Trips	Vehicle-miles	Trips	Vehicle-miles	Trips
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Professional and semiprofessional workers	97.1	94.8	97.4	96.0	97.0	94.4
Proprietors, managers, and officials:						
Farmers and farm managers	87.3	93.6	87.8	94.5	82.5	83.9
Other proprietors, managers, and officials	90.4	89.8	97.2	91.8	88.5	89.4
Store and office clerks	92.3	90.3	94.1	88.3	91.8	90.9
Traveling salesmen	100.0	100.0	100.0	100.0	100.0	100.0
Craftsmen, foremen, and skilled laborers	90.5	95.0	97.8	97.8	87.3	93.8
Operatives, semiskilled, and unskilled laborers	93.4	93.2	93.9	91.5	93.1	94.0
Protective services workers	96.8	89.0	93.9	76.9	97.7	91.8
Military personnel	76.7	88.1	100.0	100.0	76.1	87.4
Personal service workers	91.3	91.0	82.8	75.8	93.4	93.7
Retired persons	89.3	95.1	92.1	90.0	88.6	96.3
Housewives	63.0	57.5	52.0	51.5	70.0	60.3
Unemployed persons	86.9	79.4	83.3	71.1	88.9	83.3
Students	65.9	70.6	73.0	69.1	60.4	71.5
ALL OCCUPATIONS	88.0	86.4	88.0	85.5	87.9	86.9

[1] Special analysis of 24,000 trips reported on motor-vehicle-use study interview forms from Colorado, Delaware, Kansas, and Tennessee. This represented a subsample of motor-vehicle-use study interview forms.

planners and city officials for estimating the number of automobiles that may travel on a given highway if an industrial plant or shopping center is located within any specified area and for comparing with data on other modes of transportation.

The average occupancy for all trips was 1.7 persons per trip, as shown in table 14 and figure 9. For trips that were confined entirely to an incorporated place or to contiguous places, occupancy averaged 1.6 persons per trip, but when the trip was made entirely outside of incorporated places or partially

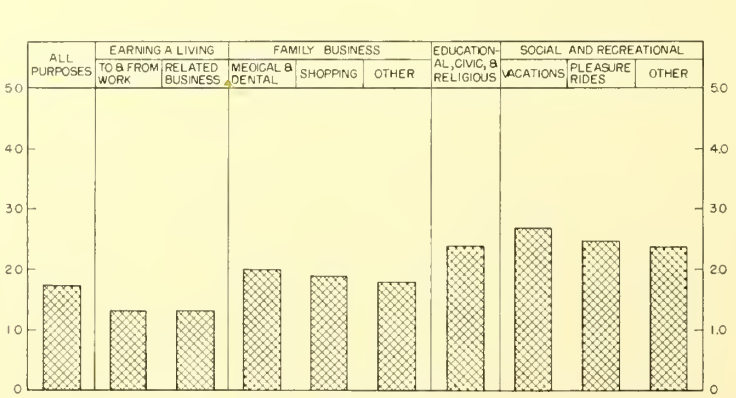

Figure 9.—Average occupancy of automobile classified by purpose of trip.

Table 17.—Proportion of travel performed by each occupational group of drivers [1]

Occupational group of driver	Proportion of travel performed by driver	Proportion of vehicle-miles driven by each occupational group as—	
		Principal operator	Other than principal operator
	Percent	*Percent*	*Percent*
Professional and semiprofessional workers	12.1	88.4	11.6
Proprietors, managers, and officials:			
Farmers and farm managers	9.9	82.6	17.4
Other proprietors, managers, and officials	10.7	91.6	8.4
Store and office clerks	11.6	90.3	9.7
Traveling salesmen	2.3	97.3	2.7
Craftsmen, foremen, and skilled laborers	16.8	95.0	5.0
Operatives, semiskilled, and unskilled laborers	16.6	92.4	7.6
Protective services workers	1.4	94.6	5.4
Military personnel	2.2	96.8	3.2
Personal service workers	1.8	90.2	9.8
Retired persons	1.1	96.0	4.0
Housewives	10.6	59.8	40.2
Unemployed persons	0.9	98.9	1.1
Students	2.0	98.6	1.4
ALL OCCUPATIONS	100.0	88.0	12.0

[1] Special analysis of 24,000 trips reported from motor-vehicle-use studies in 4 States: Colorado, Delaware, Kansas, and Tennessee. This represented a subsample of motor-vehicle-use study interview forms.

Table 18.—Average length of one-way trip by purpose of travel and day of week [1]

Day	All trips	Earning a living			Family business				Educational, civic, and religious	Social and recreational
		To and from work	Related business	All work trips	Medical and dental	Shopping	Other family business	All family business		
	Miles	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*	*Miles*
Monday	7.5	7.4	14.7	8.3	6.8	3.9	6.0	5.2	5.9	12.1
Tuesday	7.6	7.3	18.7	8.6	5.6	4.9	6.9	5.9	4.1	10.8
Wednesday	7.8	7.3	20.6	8.8	4.8	4.7	7.8	6.3	4.2	10.5
Thursday	7.0	7.4	17.3	8.3	9.3	4.4	5.5	5.3	6.3	6.7
Friday	7.7	7.0	15.6	8.0	12.3	4.3	6.5	5.7	6.0	12.9
Saturday	8.3	6.3	13.1	7.3	8.8	5.1	8.1	6.5	7.5	13.9
Sunday	10.6	8.1	16.1	9.9	7.4	4.0	8.8	7.3	4.1	18.8
ALL DAYS	8.0	7.2	16.7	8.3	7.9	4.6	7.0	6.0	5.0	13.6

[1] National automobile-use study conducted by the Bureau of the Census for Public Roads, spring 1961.

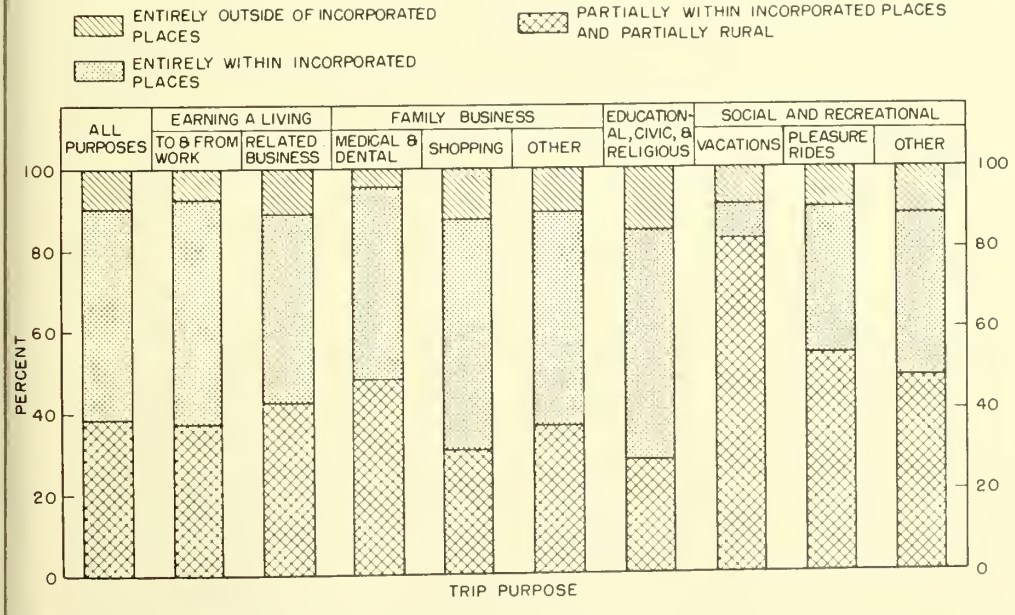

Figure 10.—Automobile trips classified by purpose of each trip and location of travel.

within an incorporated place and partially in a rural area, the average occupancy rate was 1.9 persons per vehicle. On trips made by unincorporated area residents, occupants generally averaged more persons per trip than for the trips made by residents of all incorporated places.

The average occupancy rate for trips related to earning a living was 1.3 persons per trip for all population groups combined. Residents of all incorporated places reported an average occupancy of 1.2 persons per trip, while residents of unincorporated areas reported 1.4 occupants per trip.

On trips made for medical and dental purposes, occupants averaged 2.0 per trip, and 2.2 occupants per trip by residents of unincorporated areas and 1.9 occupants per trip by residents of incorporated places were reported. Where the trip was made partially through an incorporated place and partially through a rural area, the average occupancy per trip was somewhat higher. Other trips made in connection with family business followed somewhat the same pattern as trips for medical and dental purposes, but the average occupancy rate was a little lower: 1.9 persons per trip for shopping purposes and 1.8 persons per trip for other family business.

For trips related to educational, civic, and religious purposes, occupancy averaged 2.4 persons per trip for residents of all places, and 2.6 and 2.3 for residents of unincorporated areas and incorporated places, respectively. The largest number of occupants per trip, an average of 2.7 persons was reported for vacation trips. Unincorporated area residents reported an average occupancy of 3.1 persons for vacation trips, and residents of all incorporated places reported an average occupancy of 2.7 persons per trip. For trips for pleasure rides and other social and recreational purposes, average occupancy was 2.5 and 2.4 persons, respectively.

Distribution of Automobile Trips by Purpose and Location of Travel

As shown in table 15 and figure 10 more than half of all passenger-car trips were made entirely within an incorporated place or within contiguous places. An additional 38 percent of all trips was classified as trips partially within incorporated places and partially in rural areas. These latter trips probably approximate the so-called intercity travel, although some of the trips are undoubtedly not truly intercity.

Fifty-six percent of all to-and-from-work trips was confined entirely within a single incorporated place or contiguous places. Half or more of the trips for purposes other than social and recreational was also urban in character. Eight percent of all vacation trips was taken entirely within incorporated places; the bulk of such travel was reported by California, Kansas, Illinois, and Pennsylvania. Eighty-four percent of all vacation trips involved travel both within incorporated places and in rural areas. Similarly, only a small proportion of trips for any purpose were made completely outside an incorporated place (in

Table 19.—Distribution of trips and travel by day of week for each purpose of travel [1]

Day	All purposes		Earning a living		Family business								Educational, civic, and religious		Social and recreational	
					Medical and dental		Shopping		Other family business		Total					
	Trips	Travel	Trips	Travel	Trips	Travel	Trips	Travel	Trips	Travel	Trips	Travel	Trips	Travel	Trips	Travel
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Monday	15.2	14.1	19.7	19.5	21.5	18.5	12.6	10.6	13.0	11.0	13.4	11.5	13.5	16.1	9.2	8.2
Tuesday	13.4	12.6	16.7	17.3	14.1	10.0	11.2	11.9	13.2	12.9	12.3	12.3	11.4	9.4	8.9	7.1
Wednesday	13.3	12.9	16.3	17.3	16.4	10.1	11.2	11.6	13.5	14.9	12.6	13.3	10.7	9.0	9.4	7.2
Thursday	13.4	11.7	15.9	15.8	16.1	19.0	12.1	11.6	14.0	10.8	13.2	11.8	9.9	12.7	10.6	5.2
Friday	16.5	15.8	18.7	18.1	16.0	25.0	19.9	18.8	16.2	14.9	17.9	17.2	10.5	12.6	12.9	12.2
Saturday	15.4	16.0	9.6	8.4	14.0	15.6	28.3	31.4	20.1	23.0	23.6	25.4	5.6	8.4	20.6	21.6
Sunday	12.8	16.9	3.1	3.6	1.9	1.8	4.7	4.1	10.0	12.5	7.0	8.5	38.4	31.8	28.4	39.1

[1] National automobile-use study conducted by the Bureau of the Census for Public Roads, spring 1961.

a rural area), the greatest number, 14.1 percent, being made for educational, civic, and religious purposes.

Who Does the Driving

Car manufacturers are concerned about their potential market. Thus, it follows that they need to know who drives the vehicles. For example, if the housewife does a high proportion of all the driving, manufacturers undoubtedly will want to give consideration to women when designing new automobiles.

The standard interview form for the motor-vehicle-use studies asks for the occupational group of the principal operator of a vehicle and also the occupational group of the driver for each trip. For the standard tabulations prepared from these studies, all trips and travel were classified according to the occupation of the person shown as the principal operator. To determine the proportion of trips and travel performed by each occupational group according to the principal operator and other than the principal operator, a special analysis was made of a subsample of interview forms from Colorado, Delaware, Kansas, and Tennessee. This analysis showed that 88 percent of all travel and 86 percent of all trips were performed by the persons reported as the principal operators.

Table 16 shows the proportion of travel and trips performed by the principal operator according to occupational groups for residents of all places, all unincorporated areas, and all incorporated places. Traveling salesmen performed all the trips and travel in the vehicles for which they were reported as the principal operators. Persons engaged in the protective services and those in the professional and semi-professional groups reported a very large percentage of the vehicle-miles and trips in the vehicles for which they were reported as being the principal operators. Housewives, however, performed only 63 percent of the travel and 58 percent of the trips in automobiles in which they were reported as the principal operators.

Table 17 shows the proportion of travel performed according to the occupational group

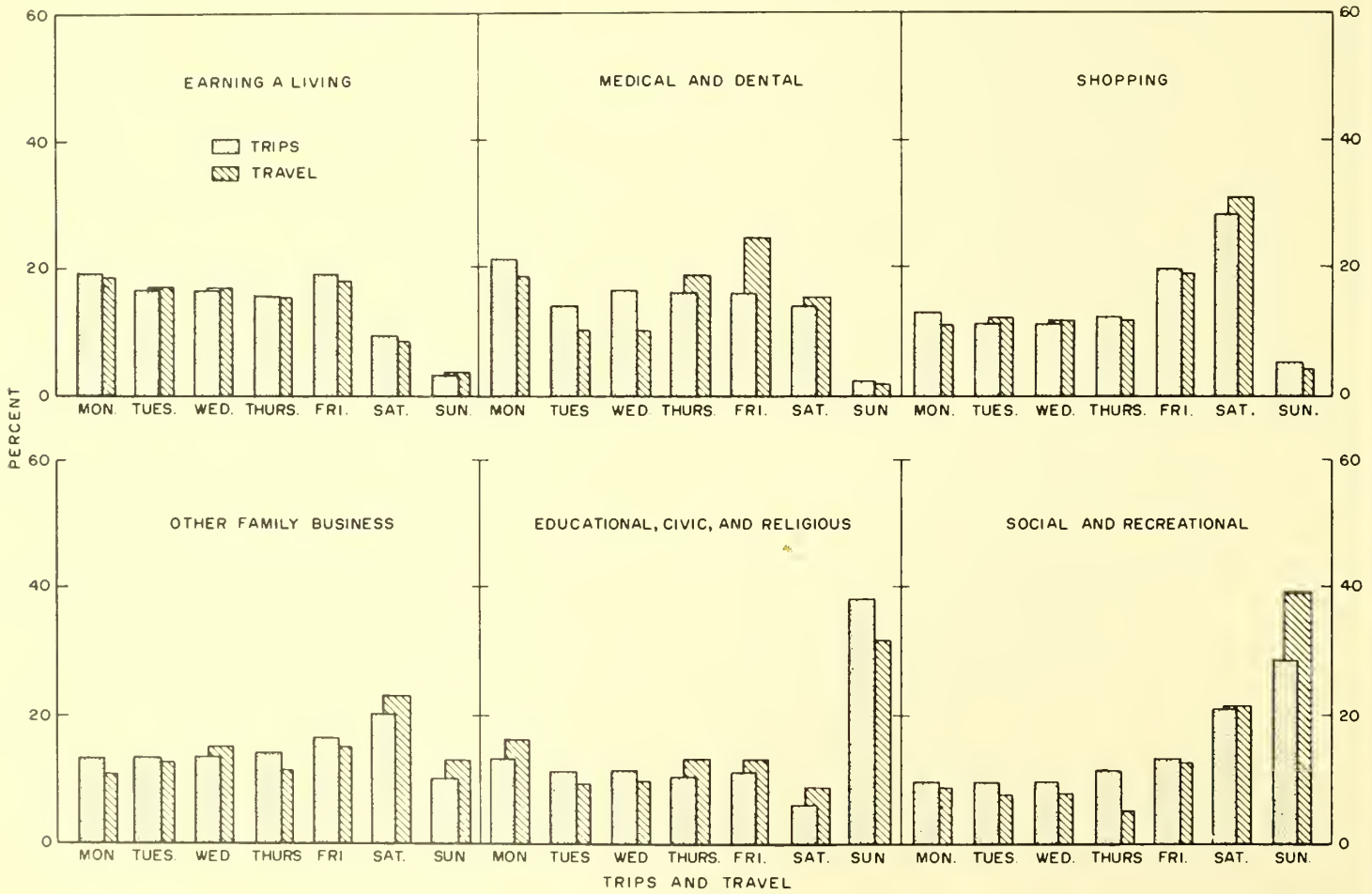

Figure 11.—Purpose of trips and travel classified for day of week.

of the driver. Also, this table shows, by occupational groups, the proportion of travel performed as the principal operator and as other than the principal operator. Skilled and unskilled laborers (including craftsmen, foremen, and operatives) performed one-third of the total travel. Persons in the protective services, traveling salesmen, military personnel, and personal service workers performed a small proportion of the total travel; they also accounted for only a small proportion of the total workers.

Housewives accounted for a high proportion of driving (40.2 percent) in automobiles for which they were not reported as the principal operators. Farmers and farm managers and professional and semiprofessional workers also reported that a high proportion of their driving was done in vehicles in which they were not reported as the principal operator.

Trips and Travel by Day of Week

The average one-way-trip length by passenger cars owned by residents as determined from the study conducted by the Census Bureau for Public Roads was 8.0 miles. This finding agrees with similar data developed from State motor-vehicle-use studies. The tabulations prepared by the Census Bureau give data showing trips and travel for each day of the week. Table 18 shows that trips made on weekdays for all purposes combined were shorter than trips made on weekends. Length of trips made on weekdays averaged 7.0 to 7.8 miles, trips made on Saturdays averaged 8.3 miles, and trips made on Sundays averaged 10.6 miles. Sunday trips made for purposes of earning a living and for social and recreational purposes tended to be longer than trips made for the same purposes on other days. For example, persons who made trips to and from work on Sundays drove an average of 8.1 miles, as compared with from 7.0 to 7.4 miles on weekdays and 6.3 miles on Saturdays.

Table 19 and figure 11 show the distribution of all trips and travel by the day of week for selected purposes of travel. The highest proportion of all trips was made on Fridays and the highest proportion of all travel on Sundays. There were wide variations in the distribution of trips and travel by day of week. The lowest proportion of all trips related to earning a living or family business was made on Sundays. The highest proportion of all trips and travel related to earning a living was made on Mondays, the next higher proportion being reported on Fridays.

Although many persons do shop during the weekdays, approximately 30 percent of all trips and travel for shopping are made on Saturdays. Because many stores are open at least some evenings during the week, it might be anticipated that a smaller proportion of trips and travel for shopping would be made on Saturday. One possible reason for the large number of shopping trips on Saturdays may be that many workers are paid on Fridays and Saturdays. Also, it is possible that the type of shopping done on Saturdays may be different from that done on the weekday shopping trips.

More than 38 percent of all trips and almost 32 percent of all travel for educational, civic, and religious purposes were made on Sunday. This naturally follows as Sunday is the principal day for trips to church. However, it would be expected that trips and travel performed for civic and educational purposes would bulk large on weekdays and would tend to pull these percentages down. Sunday was also the most popular day for social and recreational trips; 28 percent of all such trips and 39 percent of all such travel were made on this day.

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(*2*) *Automotive Transportation, Trends, and Problems,* by W. Owen, Brookings Institution, Washington, D.C., 1949, pp. 7–9.

(*3*) *Highway Statistics Summary to 1955,* by Bureau of Public Roads, 1957, table MV–201, p. 18.

(*4*) *Highway Statistics 1961,* by Bureau of Public Roads, 1963, table MV–1, p. 14.

(*5*) *Forecasts of Population, Motor-Vehicle Registrations, Travel, and Fuel Consumption,* by T. R. Todd, Public Roads, vol. 30, No. 12, Feb. 1960, pp. 266–267.

(*6*) *Highway Statistics 1961,* by Bureau of Public Roads, 1963, table MV–12, p. 19.

(*7*) *Urban Transportation—Service or Chaos,* by H. E. Davis, presented at annual meeting of California State Chamber of Commerce, Dec. 1953, p. 4 (processed).

(*8*) *U.S. Census of Population: 1960, Final Report PC(1)–1C, General Social and Economic Characteristics, United States Summary,* by the Bureau of the Census, table 94, p. 1–224.

(*9*) *United States Census of Housing: 1960, Final Report HC(1)–1, States and Small Areas, United States Summary,* by the Bureau of the Census.

Interstate System Accident Research

BY THE TRAFFIC SYSTEMS RESEARCH DIVISION BUREAU OF PUBLIC ROADS

By STANLEY R. BYINGTON, Highway Research Engineer

This article reports the initial findings of a study comparing the traffic accident experience on completed portions of the Interstate Highway System with that on nearby highways, which formerly carried the largest percentage of interstate traffic. Included are data from 16 States for more than 1,000 miles of both Interstate System and nearby highways.

Results of the comparison show that accident rates on the Interstate System are about half as great as those on nearby highways and injury and fatality rates are about one-third as great. Data also show that the Interstate Highway System produced the greatest net reduction in accident rates in the more densely populated areas and the greatest reduction in fatality rates in rural areas. As these areas normally have, respectively, the highest accident and fatality rates, the safety benefits of the Interstate Highway System in terms of accident and fatality reductions are greatest where the need is greatest.

A preliminary estimate has been made that 8,000 lives a year may be saved upon completion of the Interstate Highway System. The article emphasizes that this estimate is preliminary and is based on limited data.

Information collected on the effect of traffic volume, an analysis by manner of collisions, and the importance of access control also are summarized in this article.

Introduction

IN 1939 AND again in 1944, the Bureau of Public Roads reported to the Congress the need for a special system of interregional highways and the necessary connections through and around cities. Legislation passed in 1944 authorized such a System and legislative acts of 1956 and following years have assured the construction of this system, a 41,000-mile network, commonly called the Interstate System.

The need for this System has been expressed in terms of social, economic, and defense benefits. In addition, it has been estimated by different sources that completion of the Interstate System would save from 5,000 to 9,000 lives a year.[1] Accident data are currently being collected by a number of States in a study designed to measure the actual safety benefits that can be attributed to the completion of portions of this system. Accident experience on the Interstate System is being compared with accident experience on nearby highways, hereafter referred to as "existing highways," that Interstate System traffic formerly had to traverse. For brevity, references to data collected prior to construction of the Interstate System are labeled "before" and data collected after its construction "after." When Interstate and existing highway "after" data are combined, reference is made to the Interstate corridor.

This article, which is introductory in scope, describes the study techniques and summarizes the findings based on data collected for 1,130 miles of the Interstate System and 1,000 miles of existing highways. As additional data are collected, more extensive analyses will be made and additional reports will be prepared. Plans for future reports include a more detailed examination of property damage costs associated with accidents occurring on each highway system and the interaction of different design and traffic variables.

Summary

The following paragraphs contain a summary of the principal findings from the first group of data collected in the Interstate System Accident Research Study.

. On the average, accident rates on the Interstate System were slightly more than one-half those on nearby existing highways, either before or after the Interstate System was opened to traffic.

. Injury and fatality rates on the Interstate System were slightly more than one-third as great as those on existing highways.

. The accident and injury rates on the existing highways did not change much after the Interstate System was opened and some traffic was diverted from these highways to the Interstate System. However, fatality rates on existing highways in rural areas declined by more than one-half after opening of the Interstate System.

. The more densely populated urban areas had the greatest net reduction in accident and injury rates, as shown in the comparison of existing highways "before" and the Interstate corridor. Also, the net reduction in injury rates for rural areas followed closely that of the highly urbanized areas. Fatality rates in rural areas were reduced by 71 percent as compared with 24 percent for urban areas.

. The accident rate generally increased as traffic volume increased; this trend was particularly evident for existing highways.

. As expected, head-on, opposite-direction sideswipes, angle, and pedestrian collisions were nearly eliminated on Interstate highways. About one-third of all collisions on these highways were of the rear-end or same direction sideswipe types, and nearly all of the remaining accidents involved only a single vehicle.

. Although control of access is the most important single factor contributing to the excellent safety record of the Interstate System, there is an indication that other elements of modern highway design, such as wide medians, easy curvature and gradient, and long sight distances, are also important.

Study Techniques

In May 1960, all States except Alaska, which had no Interstate System mileage, were invited to participate in an accident study designed to measure the safety benefits that construction of the Interstate System would provide. Up to the present time, 43 States have indicated that they will participate in this research study, and 36 States are already collecting data. This article presents an analysis of the accident data compiled by the 16 States shown in figure 1. As of the closing date selected for summarizing the data, these States had advanced their studies sufficiently to permit at least a limited analysis. Eventually, it is expected that much of the Interstate System mileage plus an equivalent mileage of existing highways will be included in the study.

Study Sections

Mileage on three types of highways was included in this study of highways: Interstate, existing, and control. Existing highways are those highways within the Interstate System corridors that formerly carried the greatest proportion of the present Interstate traffic. Figure 2 illustrates Interstate and existing highway study sections in Iowa.

Whenever an Interstate highway replaced an existing highway on the same location, an effort was made to select a control highway for

[1] *Future Highways and Urban Growth*, by Wilbur Smith & Associates under commission from the Automobile Manufacturers Association. Feb. 1961, p. vii. *Life Saving Benefits of the Interstate System*, by Charles W. Prisk. PUBLIC ROADS, vol. 31, No. 11, Dec. 1961, pp. 219-220.

Table 1.—Number of study sections included in the analysis, length of study sections, and vehicle-miles of travel, by State and type of highway

Types of highways	Arizona	Florida	Georgia	Indiana	Iowa	Kansas	Minnesota	Missouri	New Mexico	New York	North Dakota	Rhode Island	Utah	Vermont	Virginia	Wyoming	Total
Existing highways "before":																	
Number of study sections	6	8	(1)	2	22	7	1	4	35	114	7	4	2	1	8	8	229
Length of sections—miles	68.0	22.5	(1)	11.7	67.7	59.1	8.2	25.3	248.4	340.8	32.8	19.1	16.8	7.9	11.1	61.1	1,000.5
Vehicle-miles—100,000	1,440	716	(1)	316	2,513	1,399	293	1,663	10,167	13,645	663	714	2,122	87	420	1,143	37,301
Existing highways "after":																	
Number of study sections	------	8	8	2	22	5	1	4	5	139	6	4	1	1	8	4	218
Length of sections—miles	------	22.5	35.2	13.4	67.7	39.2	8.2	25.1	29.0	424.7	30.6	19.1	8.2	7.9	11.1	11.1	753.0
Vehicle-miles—100,000	------	490	4,453	340	472	335	87	729	53	29,692	93	412	287	87	270	119	37,919
Control highways "before":																	
Number of study sections	6	------	------	------	------	------	------	------	7	------	1	------	1	------	------	------	15
Length of sections—miles	69.3	------	------	------	------	------	------	------	33.5	------	2.5	------	9.0	------	------	------	114.3
Vehicle-miles—100,000	1,318	------	------	------	------	------	------	------	1,717	------	12	------	171	------	------	------	3,218
Control highways "after":																	
Number of study sections	6	------	------	------	------	------	------	------	7	------	1	------	1	------	------	------	15
Length of sections—miles	69.3	------	------	------	------	------	------	------	33.5	------	2.5	------	9.0	------	------	------	114.3
Vehicle-miles—100,000	793	------	------	------	------	------	------	------	1,383	------	1.3	------	36	------	------	------	2,225
Interstate highways, type 5: [2]																	
Number of study sections	6	9	------	2	2	7	1	4	------	13	6	2	2	1	2	8	65
Length of sections—miles	68.0	21.3	------	11.7	27.6	59.1	8.0	29.6	------	130.8	30.6	8.1	16.9	7.5	12.2	61.7	493.1
Vehicle-miles—100,000	809	513	------	477	208	626	248	1,575	------	9,013	587	234	478	140	174	505	15,587
Interstate highways, type 6: [3]																	
Number of study sections	------	------	5	1	------	------	------	------	9	23	------	------	------	------	------	------	38
Length of sections—miles	------	------	32.2	6.4	------	------	------	------	52.5	310.1	------	------	------	------	------	------	401.2
Vehicle-miles—100,000	------	------	8,294	806	------	------	------	------	1,054	50,579	------	------	------	------	------	------	60,733
Interstate highways, type 7: [4]																	
Number of study sections	------	------	1	------	3	------	------	------	26	------	------	1	------	------	------	------	31
Length of sections—miles	------	------	5.2	------	24.7	------	------	------	195.9	------	------	9.4	------	------	------	------	235.2
Vehicle-miles—100,000	------	------	1,849	------	298	------	------	------	8,213	------	------	755	------	------	------	------	11,115

[1] Interstate System study sections were opened to traffic in 1958. No information is presented for comparable existing "before" study sections because accident data were not available prior to 1958.

[2] Study sections were fully improved to Interstate standards. Data included on sections in which the roadway, structures, ramps, guardrails, etc., were accepted from the contractor by the State and opened to traffic. Such work as seeding, signing, striping, and other minor items may or may not havebeen completed.

[3] Study sections include highways that were under construction or completed before July 1, 1956. Such facilities may have substandard shoulders, ramps, median, or other characteristics that may or may not be scheduled for improvement under the current Interstate program.

[4] Study sections include highways that had some minor at-grade intersections. Facilities were not in accordance with AASHO Geometric Design Standards for the National System of Interstate and Defense Highways.

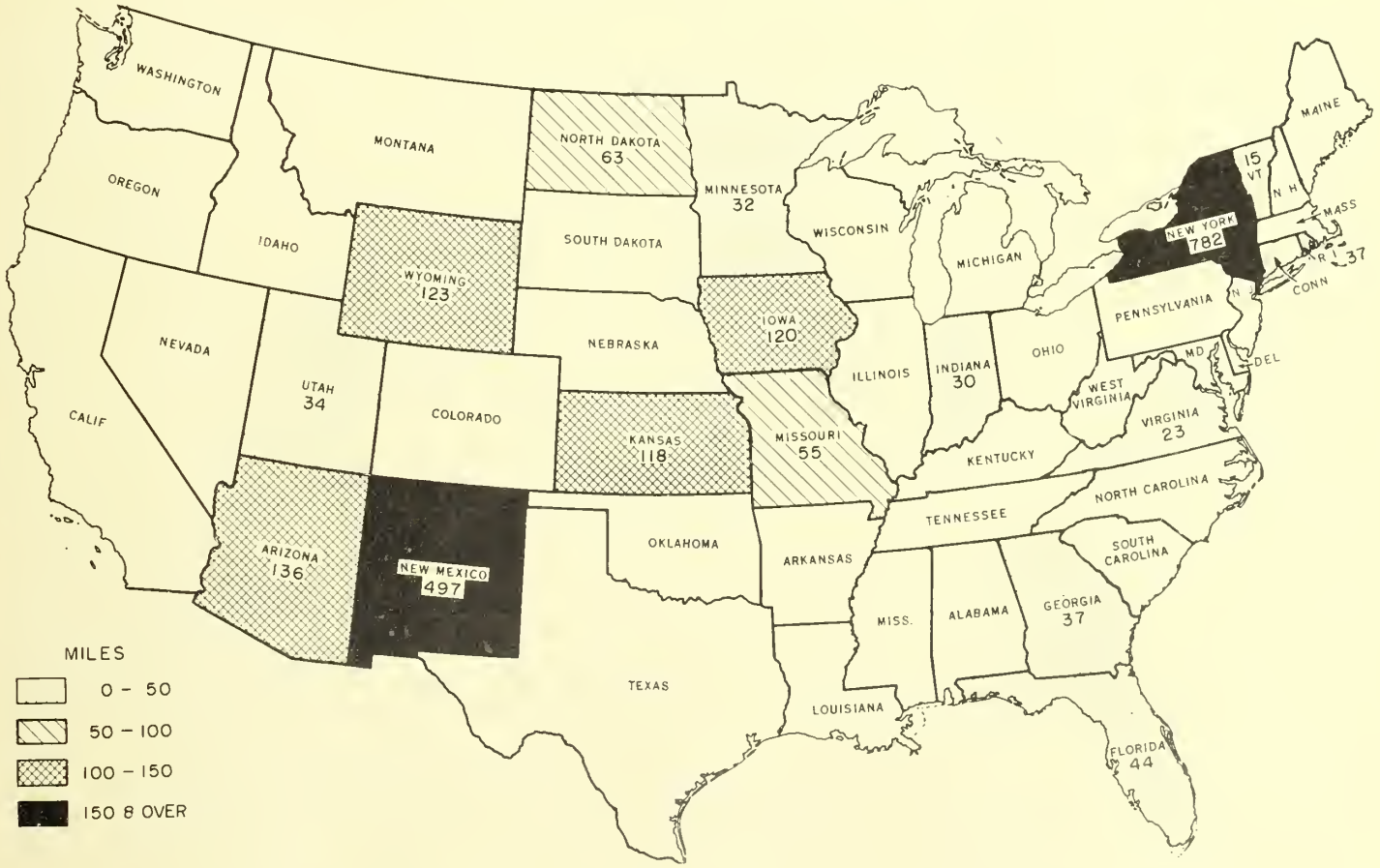

Figure 1.—Mileage of highway sections included in study. (Existing "before" plus Interstate.)

study. Control highways are those routes in reasonable proximity to the obliterated highways that have design, roadside development and average daily travel characteristics similar to the replaced sections. All three types of highways were divided into study sections for purposes of analysis. These sections were homogeneous with respect to average daily traffic, roadside development, type of area, and number of traffic lanes.

Table 1 shows the number of study sections, their length, and vehicle-miles of travel applicable to the study sections for each of the 16 States for which data are analyzed in this article. Travel data are the summation of vehicle-miles applicable to all study sections for a different number of years before or after the opening of Interstate System sections. A total of 1,000 miles of existing highways, 114 miles of control highways, and 1,130 miles of Interstate highways constituted the lengths of sections under study. A more general geographical distribution of study section mileages would have been desirable in this analysis, as the findings were influenced considerably by data compiled in two States—New Mexico and New York.

Figure 3 is a facsimile of the form used by the States in compiling all of the basic information for the study. One year of data for each study section was recorded on the form.

Highway Data

The highway data collected for each study section consisted of length of section, degree of access control, roadside development, number of at-grade intersections, type of area, and type of highway as shown in items 6–10 of figure 3. The length for each section was determined to the nearest one-tenth mile through the use of "as-built" construction plans, road mileage logs, or odometer readings.

Traffic Data

Average daily traffic volumes were determined for each study section by one of the following described methods: (1) one continuous 24-, 48-, or 96-hour count; (2) four continuous 24-hour counts taken at four different times during the year; or (3) one continuous 7-day count plus one 24-hour count at another time of the year. In the first method, a section's average daily traffic was determined to the nearest 100 vehicles by adjusting the measured volumes for seasonal and daily variations through control station counts. When either of the other two methods were employed, average daily traffic was determined to the nearest 100 vehicles by an averaging process of the measured volumes.

The data from regular traffic counting programs were utilized to the extent possible. Where the regular counting programs did not meet the requirements of the study or no counting programs were in progress, special counts had to be made. The scope of the traffic counting operation may be reduced in the future when data on seasonal fluctuations and general trends appear to be fairly well established.

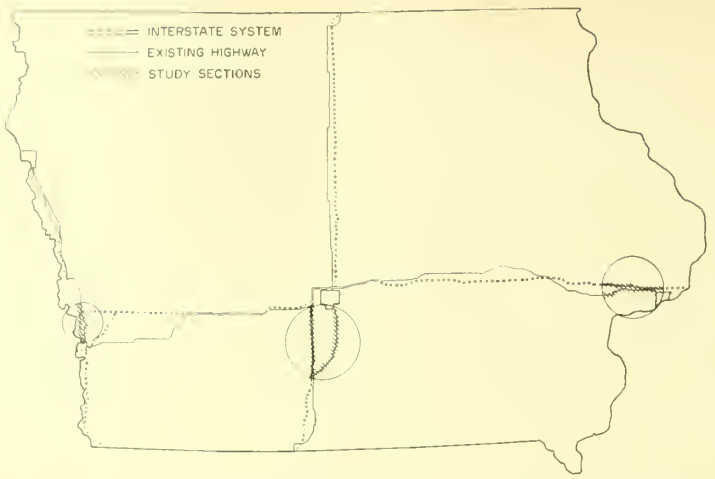

Figure 2.—Illustration of Interstate System and existing parallel highway study sections (encircled) in Iowa. (Broken lines indicate proposed location of Interstate System.)

Accident Data

Wherever possible, accident and related data were obtained for existing highways 4 to 6 years before the opening of the Interstate highway. Accident data obtained for each study section included the total number of accidents reported to State or local authorities, number of persons injured, number of persons killed, and estimated costs of property damage. The number of accidents reported were distributed by types of collision. Accidents that occurred at intersections (at grade) and interchanges, on frontage roads, and on Interstate grade-separated crossroads not having interchanges were included in the study. At interchanges, crossroad accidents were included if they occurred within the limits of 100 feet "outboard" of the ramp terminals. On Interstate grade-separated crossroads not having interchanges, accidents were included when they occurred within 100 feet of the structure or on the structure. The different subgroupings of estimated property damage costs and types of collisions are shown as items 13 and 16 of figure 3. Most accident data were obtained from the States' central accident record files. Depending on the State, these files are maintained by the State highway patrol, the State records division, or a State agency or department of motor vehicles, public safety, law enforcement, or revenue. Other accident data were obtained from records of the city police, county sheriff offices, and toll authorities.

Accident data are being collected for all study sections on a continuing basis. Analyses of these accident data will help to define possible trends that may develop as adjacent sections of the Interstate System are completed and traffic volumes increase.

Property Damage Costs

All accidents for each study section were classified into one of four categories, based on property damage cost information shown on accident records of State or local authorities: (1) less than $100, (2) $100 to $499, (3) $500 or over, and (4) property damage cost not known.

Analysis of Data

The data lent itself to two general types of analysis: group and individual. In the group

Table 2.—Comparison of accidents, injuries, and fatalities occurring on existing highways "before" and "after," with those on Interstate highways.

Item of comparison	Existing highways		Interstate highways only	Interstate highways plus existing highways "after"
	"Before"	"After"		
Number of accidents	8,892	9,881	11,829	21,710
Vehicle-miles—millions	3,730	3,792	8,744	12,536
Accidents per 100 million vehicle-miles	238	261	135	173
Difference in accident rates [1]		23	−103	−65
Percentage change in accident rates [1]		9.7	−43.3	−27.3
Number of injuries	4,968	5,372	4,343	9,715
Injuries per 100 million vehicle-miles	133	142	50	78
Difference in injury rates [1]		9	−83	−55
Percentage change in injury rates [1]		6.8	−62.4	−41.4
Number of fatalities	362	193	247	440
Fatalities per 100 million vehicle-miles	9.7	5.1	2.8	3.5
Difference in fatality rates [1]		−4.6	−6.9	−6.2
Percentage change in fatality rates [1]		−47.4	−71.1	−63.9

[1] Numerical difference or percentage change in rates when compared to existing highways "before."

analysis, comparisons of accident data were made between highway sections having similar traffic volumes. For example, accidents on existing highway sections having traffic volumes between 4,000 and 8,000 vehicles per day were compared with those on Interstate sections, regardless of location, which carried similar traffic volumes. For the individual analysis, comparisons were made only on accident data collected for existing highways and the nearby Interstate study sections. For example, accidents on existing highway sections were compared with those on parallel Interstate sections regardless of the design characteristics of the two types of highways.

Within the two types of analysis, accident rate comparisons were possible for (1) existing highway sections "before" compared with Interstate System sections; (2) existing highway sections "before" compared with existing highway sections "after"; and (3) existing highway sections "before" compared with Interstate plus existing highway sections "after." When sufficient control section data are obtained, a comparison can be made between accidents on control highway sections and existing highway sections "before" with accidents on control highway sections "after" and Interstate System sections. All of the preceding comparisons were included in the analysis of data described in subsequent sections of this article, except control highway comparisons. In the analysis in this article, control highway section data were reported by only 4 of the 16 States, and the limited mileage did not provide a basis for meaningful comparisons. In the analysis consideration was given to such variables as average daily traffic volume, type of study area, degree of access control, number of traffic lanes, and types of accident or collision.

Accident, Injury, and Fatality Rates

In table 2, the number of accidents and the number of persons injured and killed are related to travel on the basis of 100 million vehicle-miles. This accident rate for Interstate highways was 135 accidents per 100 million vehicle-miles, as compared to 238 and 261 accidents per 100 million vehicle-miles respectively, for existing highways "before" and existing highways "after." Thus, the accident rates on Interstate highways were 43 and 48 percent below those of the existing highways "before" and "after," respectively.

Similar comparisons of injury and fatality rates show even greater percentage reductions in these rates on the Interstate highways. Fatality rates on existing highways were reduced 47 percent after the Interstate highways were opened to traffic. The rate of 9.7 fatalities per 100 million vehicle-miles on existing highways "before" was nearly double

ITEM			DO NOT USE THIS SPACE
1. STATE			1 2
2. SECTION NO. *(1 to 999)*			3 4 5
3. YEAR *(12 months ending)*			6 7
HIGHWAY DATA			8
4. TYPE OF SECTION *(Check one box only)*			
EXISTING HIGHWAY	1.	BEFORE INTERSTATE HIGHWAY OPENED TO TRAFFIC	
	2.	AFTER INTERSTATE HIGHWAY OPENED TO TRAFFIC	
CONTROL SECTION	3.	BEFORE INTERSTATE HIGHWAY OPENED TO TRAFFIC	
	4.	AFTER INTERSTATE HIGHWAY OPENED TO TRAFFIC	
INTER-STATE HIGHWAY	5.	FULLY IMPROVED TO INTERSTATE STANDARDS	
	6.	IMPROVED SUBSTANTIALLY TO INTERSTATE STANDARDS	
	7.	FULLY OR SUBSTANTIALLY IMPROVED TO INTERSTATE STANDARDS EXCEPT THAT ONE OR MORE INTERSECTIONS ARE AT-GRADE	
5. COMPARABLE SECTION NUMBER OR NUMBERS			9 10 11
IF INTERSTATE HIGHWAY IS ON SAME LOCATION AS EXISTING HIGHWAY, CHECK HERE		1 ☐	12
6. LENGTH OF SECTION *(Miles and tenths)*			13 14 15
7. DEGREE OF ACCESS CONTROL (AASHO DEFINITION) *(Check one)*		1 ☐ FULL 2 ☐ PARTIAL 3 ☐ NONE	16
8. ROADSIDE DEVELOPMENT AND INTERSECTIONS *(Approximate number of)*			17 18 19
COMMERCIAL OR BUSINESS ESTABLISHMENTS HAVING DIRECT ACCESS TO THE HIGHWAY			
AT-GRADE INTERSECTION			20 21
9. TYPE OF AREA *(Check one)*			22
1.	WITHIN A PLACE OF 50,000 POPULATION OR OVER		
2.	WITHIN A PLACE OF 5,000 - 50,000 POPULATION		
3.	WITHIN A PLACE OF LESS THAN 5,000 POPULATION		
4.	WITHIN SUBURBAN FRINGE OF 50,000 POPULATION OR OVER		
5.	WITHIN SUBURBAN FRINGE OF LESS THAN 50,000 POPULATION		
6.	RURAL		
10. TYPE OF HIGHWAY *(Check one)*			23
1. 2-LANE	4. 4-LANE DIVIDED	7. PAIR OF ONE-WAY STREETS	
2. 4-LANE UNDIVIDED	5. 6-LANE DIVIDED	8. OTHER *(Specify)*	
3. 5 OR MORE LANE UNDIVIDED	6. 8 OR MORE LANE DIVIDED		
TRAFFIC DATA			24 25 26 27
11. AVERAGE DAILY TRAFFIC *(To nearest 100 vehicles)*			
ACCIDENT DATA			28 29 30
12. TOTAL NUMBER OF ACCIDENTS			
13. NUMBER OF ACCIDENTS WITH PROPERTY DAMAGE			31 32 33
LESS THAN $100			
$100 - $499			34 35 36
$500 OR OVER			37 38 39
AMOUNT OF PROPERTY DAMAGE NOT KNOWN			40 41 42
14. TOTAL NUMBER OF PERSONS INJURED			43 44 45
15. TOTAL NUMBER OF PERSONS KILLED			46 47
16. MANNER OF COLLISION - NUMBER OF ACCIDENTS			48 49 50
REAR-END OR SIDESWIPE, SAME DIRECTION			
HEAD-ON OR SIDESWIPE, OPPOSITE DIRECTION			51 52 53
ANGLE COLLISION			54 55 56
COLLISION WITH PEDESTRIAN			57 58 59
OTHER COLLISION			60 61 62
NON-COLLISION			63 64 65
OTHER AND NOT KNOWN			66 67 68

Figure 3.—Form used in tabulating accident data for each study section.

Table 3.—Mean rates and ranges in rates of accident occurrence, based on 2 methods of computation

Item of comparison	Summation method [1]			Average method [2]		
	Existing highways		Interstate highways	Existing highways		Interstate highways
	"Before"	"After"		"Before"	"After"	
	MEAN RATE, ALL STATES			MEAN RATE, ALL STUDY SECTIONS [2]		
Accidents per 100 million vehicle-miles	238	261	135	319	344	147
Injuries per 100 million vehicle-miles	133	142	50	174	201	62
Fatalities per 100 million vehicle-miles	9.7	5.1	2.8	11.0	9.3	3.8
	RANGE IN RATES, BY STATE			RANGE IN RATES, BY STUDY SECTION		
Accidents per 100 million vehicle-miles	114–719	115–819	66–208	0–3,914	0–3,392	0–1,142
Injuries per 100 million vehicle-miles	82–347	35–269	36–143	0–2,740	0–2,740	0– 571
Fatalities per 100 million vehicle-miles	0–16.6	0–18.9	0–9.9	0– 449	0– 276	0– 73

[1] Total accidents, injuries, and fatalities occurring on each type of highway were divided by total travel on the respective highways.
[2] Rates were determined by averaging the computed rates for each year of study section data.

Table 4.—Standard error of estimates for comparisons of accidents and injury rates by years and accident rates by average daily traffic volumes

Figure number and title	Standard error of estimate (accidents or injuries per 100 million vehicle-miles)		
	Existing highways "before"	Existing highways "after"	Interstate System highways
Figure 5: Accident rates by years before and after opening of the Interstate System	19.2	7.9	8.1
Figure 5: Injury rates by years before and after opening of the Interstate System	5.5	5.6	3.6
Figure 10: Accident rates by average daily traffic volumes before and after opening of the Interstate System	118.4	66.3	48.1

that of the national average. The generally poor performance of the existing highways undoubtedly was one of the major factors considered in the early scheduling of Interstate System construction.

The question may be asked, What is the total benefit in terms of accident reduction in the Interstate System corridor? To answer this question, the number of accidents occurring on the Interstate System and on existing highways "after" were combined, as shown in table 2. Similarly, the vehicle-miles of travel were combined and a rate computed. This rate of 173 accidents per 100 million vehicle-miles established for the Interstate corridor indicates a decrease of 27 percent or 65 accidents per 100 million vehicle-miles when compared with the accident rate for existing highways "before." In other words, for every 100 million miles of travel in the Interstate System corridor, there were 65 fewer accidents than there would have been in the same corridor if the Interstate System had not been built and if all travel had been confined to existing highways. The foregoing comparison assumes that the accident rate on existing highways would remain constant if the Interstate System were not built, an unlikely assumption as the accident rate tends to increase with increases in traffic volumes (figs. 4 and 10).

Rate Calculation Methods

Two approaches were used in calculating the rates for accidents and injuries and fatalities. In the summation method, the mean accident, injury, and fatality rates were computed by dividing total accidents, injuries, and fatalities occurring on each type of highway by total vehicle-miles traveled on the respective highways. This method was used in determining the rates shown in table 2. Readers are cautioned against general application of these data because States have studied only a limited mileage. Also, a multitude of variables were present in each of the computed rates, such as average daily traffic volumes, number of traffic lanes, rural-urban characteristics, degree of access control, and roadside development.

In the average method, the smallest data breakdown possible was employed as a single observation; that is, data for a single study section for 1 year. This method also has certain limitations. Although study sections are located in similar areas and have similar design characteristics, their average daily traffic volumes may vary throughout the year. Furthermore, the varying amount of travel on each study section causes an imbalance between observations.

As indicated in table 3, the accident rate on the Interstate System was 135 accidents per 100 million vehicle-miles. This rate was computed on the basis of total accidents and total travel on all Interstate System sections (summation method). When each year of data for each study section of the Interstate System was treated separately and the individual accident rates averaged, the resultant accident rate was 147 accidents per 100 million vehicle-miles (average method). The difference in the mean accident rates for the preceding example seems relatively small. However, if accidents on existing highways "before" had been selected for comparison, the difference would have been considerably greater.

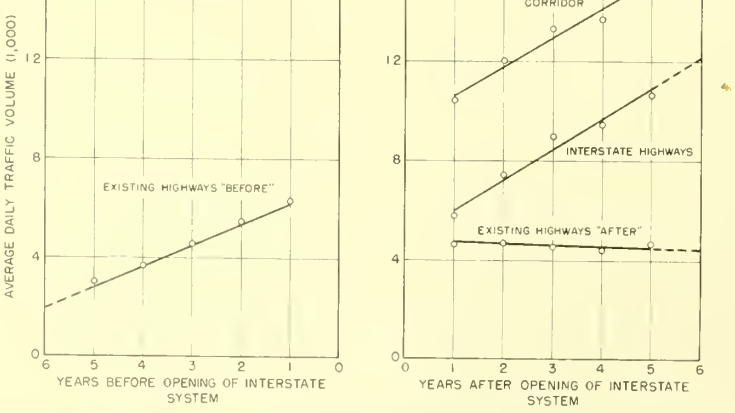

Figure 4.—Average daily traffic volumes, by years, for highway study sections before and after opening of Interstate System.

Statistical reliability

As an indication of the limitations of both methods, table 3 shows the range of accident rates by State for the summation method of computation and the range of rates by study section for the average method. The range of rates indicates that careful consideration must be given to the many design and traffic variables present for the different highway types before generalizations can be made concerning accident experience. However, on the basis of the mean rates and statistical significance curves, a logical conclusion is that the

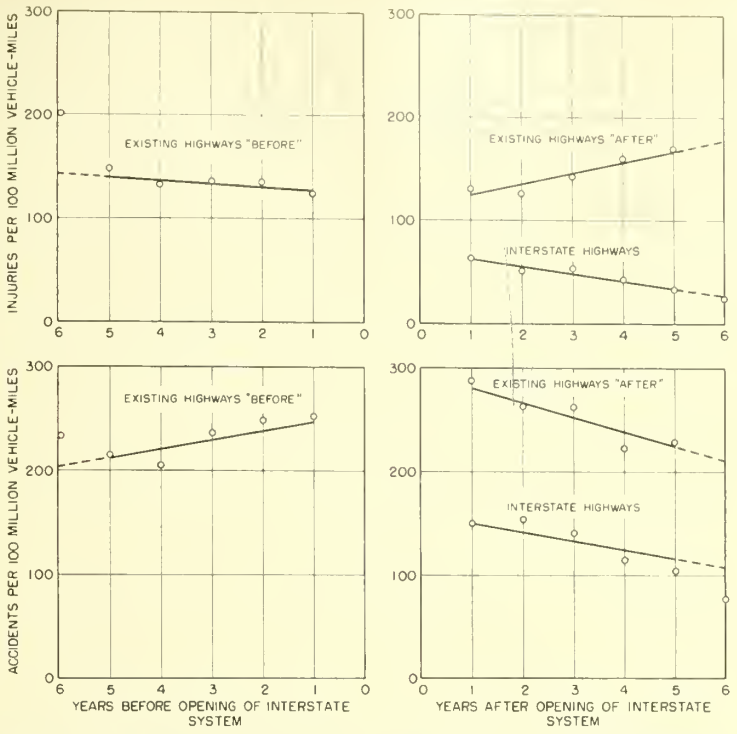
INJURIES PER 100 MILLION VEHICLE-MILES
ACCIDENTS PER 100 MILLION VEHICLE-MILES
EXISTING HIGHWAYS "BEFORE"
EXISTING HIGHWAYS "AFTER"
INTERSTATE HIGHWAYS
YEARS BEFORE OPENING OF INTERSTATE SYSTEM
YEARS AFTER OPENING OF INTERSTATE SYSTEM

Figure 5.—Accident and injury rates, by years, for highway study sections before and after opening of Interstate System.

accident rates on the same section of highway. However, there is some justification for their use because highways of the Interstate System corridor carry most of the traffic formerly carried by existing highways "before." Furthermore, if the accident rate does rise as traffic volumes increase to the highway's design capacity, as shown to be true in subsequent discussion of figure 10, then the combined "after" accident data on the Interstate System would be expected to be on the high side. As shown in figure 4, the availability of Interstate highways has generated additional travel.

Figure 4 shows an upward trend in average daily traffic volumes on the existing highways before the Interstate highways were opened to traffic. This growth averaged 17 percent annually in the 2 years before the opening of the Interstate System sections, as compared to only 4 percent each year for highways, generally. This is to be expected because sections selected for initial Interstate construction were probably those indicating the poorest performance because of traffic overloads.

After the Interstate System was opened to traffic, an initial drop occurred in average daily traffic volumes on the existing highways, this volume changed little thereafter. As

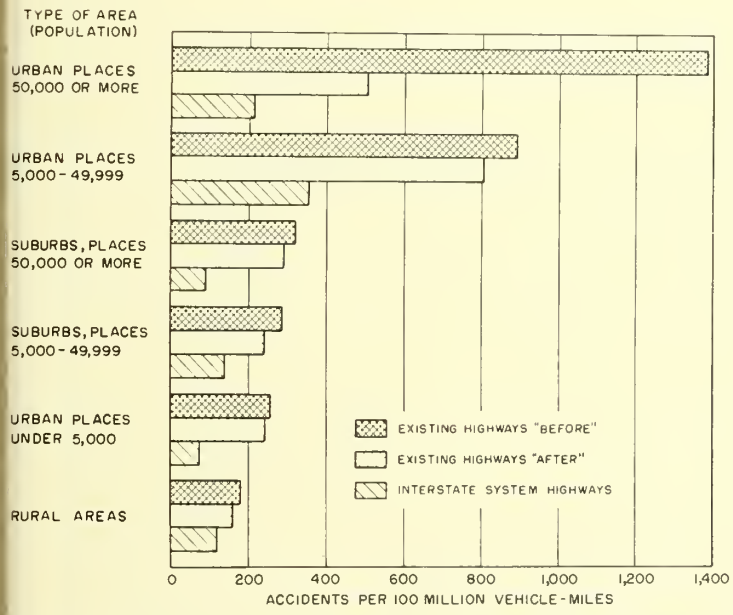

Figure 6.—Accident rates by type of area and class of highway study sections.

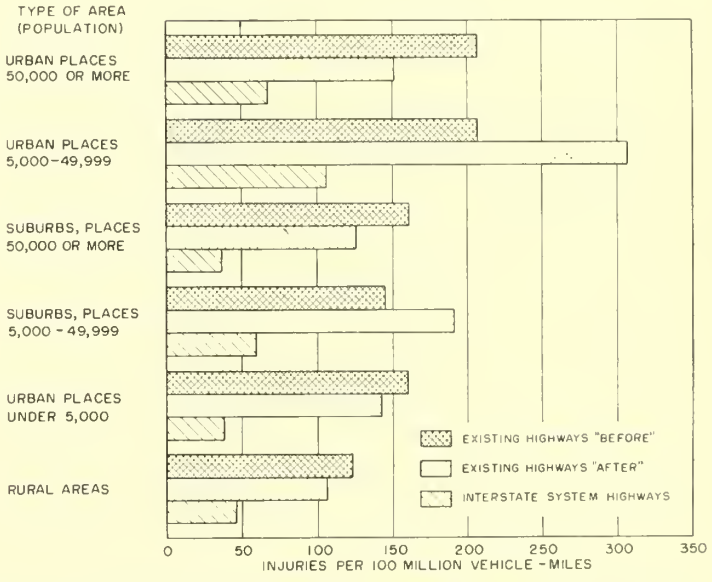

Figure 7.—Injury rates by type of area and class of highway study sections.

combined Interstate System and existing highways "after" are safer than the existing highways "before." [2]

As the accident and injury rates on the "before" and "after" sections of existing highways were nearly the same, all the benefits of accident and injury reduction were traceable to the use of Interstate highways. A Chi Square test indicates that the accident, injury, and fatality rates for the Interstate corridor were significantly lower that the rates for existing highways "before" at the 5-percent level of significance. In contrast, a comparison of rates for the existing "before" and existing "after" sections by means of the Chi Square test curve indicates that only the fatality rate is significantly different.

Readers may well question the use of the statistical curves developed by Michaels as they were designed to test before and after might be expected, traffic on the Interstate highways increased much more rapidly in absolute volumes than on the existing highways before the Interstate System was opened to traffic. Also of interest is the growth in traffic in the Interstate corridor; that is, the combined traffic on the Interstate highways and on the existing highways "after." The absolute yearly increase in traffic in the Interstate corridor was greater after the Interstate System was opened to traffic than on the existing highways "before." However,

[2] *Two Simple Techniques for Determining the Significance of Accident-Reducing Measures*, by Richard M. Michaels PUBLIC ROADS, vol. 30, No. 10 Oct. 1959, pp. 238-239.

Table 5.—Number of accidents, injuries, and fatalities before and after the opening of Interstate System study sections, by type of area

Type of area (population)	Existing highways "Before"	Existing highways "After"	Interstate System only	Interstate System corridor [1]	Existing highways "Before"	Existing highways "After"	Interstate System only	Interstate System corridor [1]	Reduction [2] Number	Reduction [2] Percent
	NUMBER OF ACCIDENTS				ACCIDENTS PER 100 MILLION VEHICLE-MILES					
Urban places, 50,000 or more	402	2,102	2,226	4,328	1,386	505	213	294	1,092	78.7
Urban places, 5,000–49,999	1,245	1,475	708	2,183	892	805	357	572	320	35.9
Suburban fringe, places of 50,000 or more	291	641	538	1,179	321	289	91	145	176	54.8
Suburban fringe, places under 50,000	1,353	1,586	759	2,345	287	239	140	195	92	32.1
Urban places, under 5,000	591	1,138	93	1,231	258	242	74	207	51	19.8
All urban places	3,882	6,942	4,324	11,266	404	355	173	253	151	37.4
Rural areas	5,010	2,939	7,505	10,444	181	160	120	129	52	28.7
	NUMBER OF INJURIES				INJURIES PER 100 MILLION VEHICLE-MILES					
Urban places, 50,000 or more	60	635	708	1,343	207	152	68	91	116	56.0
Urban places, 5,000–49,999	289	562	210	772	207	307	106	202	5	2.4
Suburban fringe, places of 50,000 or more	146	281	217	498	161	126	37	61	100	62.1
Suburban fringe, places under 50,000	684	1,266	317	1,583	145	191	59	131	14	9.7
Urban places, under 5,000	368	674	48	722	160	143	38	121	39	24.4
All urban places	1,547	3,418	1,500	4,918	161	175	60	110	51	31.7
Rural areas	3,421	1,954	2,843	4,797	123	106	46	59	64	52.0
	NUMBER OF FATALITIES				FATALITIES PER 100 MILLION VEHICLE-MILES					
Urban areas	49	107	66	173	5.1	5.5	2.6	3.9	1.2	23.5
Rural areas	313	86	181	267	11.3	4.7	2.9	3.3	8.0	70.8

[1] Existing highways "after" plus Interstate System.
[2] Existing highways "before" rate minus Interstate System corridor rate.

the percentage increases were less, ranging from 8 to 11 percent each year.

Accident Experience by Years

In any attempt to predict safety benefits that will result from construction of the Interstate System, the variation in accident and injury rates over time must be considered for the different types of highways. Figure 5 shows the accident and injury rates by years for each of the highway systems included in the study. Fatality rates are not shown accordingly because of the limited fatality data available.

Table 6.—Comparison of the effect of access control on accident and injury rates in urban and rural areas, from 2 studies

Highway access control	Urban areas: Controlled-Access Study [1]	Urban areas: Interstate System Accident Study	Rural areas: Controlled-Access Study [1]	Rural areas: Interstate System Accident Study
	ACCIDENTS PER 100 MILLION VEHICLE-MILES			
Full control of access	186	161	151	122
Partial control of access	496	264	211	94
No control of access	526	380	322	169
	FATALITIES PER 100 MILLION VEHICLE-MILES			
Full control of access	2.0	2.5	3.3	2.3
Partial control of access	4.6	3.3	6.1	6.6
No control of access	4.0	5.5	8.7	8.4

[1] *Federal Role in Highway Safety*, House Document 93, 86th Cong., 1st sess., 1959, p. 58.

Trend lines illustrated in figure 5 were developed by the Least Squares method, assuming a straightline trend. It is recognized that a straight line of regression does not always satisfactorily describe accident and injury rate trends. However, the straightline relationship appears reasonable because of the low standard errors of estimate shown in table 4 for the three types of highways.

The accident and injury trend lines in figure 5 reveal relationships described in the following paragraphs:

An upward trend in accident rates is shown for existing highways "before," but the injury rates show a slight decline. Increasing traffic congestion and corresponding decreases in average speeds may have accounted for the declining injury rate.

After the opening of Interstate highways, the accident rates on existing highways "after" declined whereas the injury rates increased.

Most noteworthy are the lower accident and injury rates on the Interstate System as compared to those on existing highways "before" and "after," and the declining rates of accidents and injuries on Interstate highways in the years following the opening of these facilities.

To determine the influence of certain design and traffic variables on accident and injury rates, subsequent sections of this article are devoted to accident comparisons made on the basis of type of area, degree of access control, and average daily traffic volumes.

Accident and Injury Rates, by Type of Area

Accident and injury rates for three types of highways located in urban places of different population sizes and in rural areas are shown in figures 6–7. Both accident and injury rates for Interstate System highways were lower than those for existing highways in rural areas and in urban areas, regardless of the population size of urban places. Accident rates were also less for the existing highways after the Interstate highways were opened to traffic. Injury rates declined on existing highways after construction of Interstate routes, except in urban places having populations from 5,000 to 50,000; no reason is apparent for the increase in injury rates in these places.

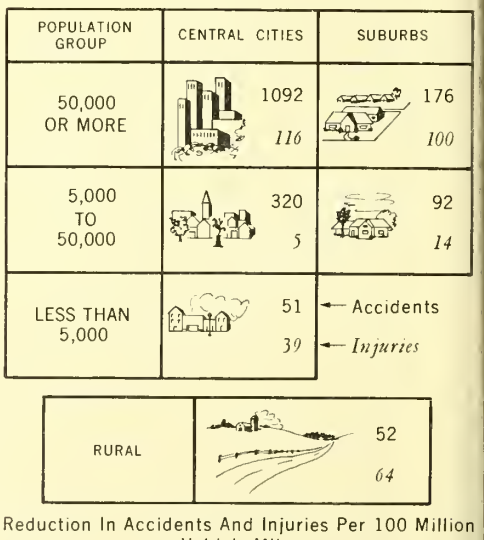

Figure 8.—Reduction in accident and injury rates, by type of area, after Interstate System study sections were opened to traffic.

The reduction in accident and injury rates brought about by the construction of Interstate System highways is evident when the rates for existing highways "before" are compared with those for the Interstate corridor (Interstate highways plus existing highways "after"). Such a comparison is given in table 5. The numerical values in figure 8 represent the reduction, after the Interstate highways were built, in accidents and injuries per 100 million vehicle-miles of travel. The greater the population density, the greater the benefits in accident reduction. Injury rate reductions were greatest in the large metropolitan areas and in rural areas. The beneficial effects that Interstate construction had on the fatality rates are also shown in table 5. In rural areas, fatality rates dropped from 11.3 to 3.3, a reduction of 8.0 fatalities per 100 million vehicle-miles. In urban areas, the net reduction was 1.2 fatalities per 100 million vehicle-miles.

On the basis of data collected in this study, valid estimates cannot be made as to the possible number of lives that will be saved when the Interstate System is completed. The reliability of these data is questioned because sections of the Interstate may have been programed for early construction in areas where existing highways had poorest performance and high fatality rates. Preliminary but unpublished data for 26 States and the District of Columbia collected for the "Interstate System Traveled-Way Study" tend to support these conclusions on Interstate programing. This latter study is a cooperative undertaking of all State highway departments and the Bureau of Public Roads. Data collected for the study reported here and the traveled-way study are compared in the following paragraphs.

In the present study, the fatality rates for 1,000 miles of existing highways "before" were 5.1 deaths per 100 million vehicle-miles in urban areas and 11.3 in rural areas. All of this mileage has now been superseded by Interstate System improvements. In the traveled-way study, 17,000 miles of highways planned to be relieved of traffic by future Interstate construction had comparable fatality rates of 3.7 deaths per 100 million vehicle-miles in urban areas and 7.0 in rural areas. By combining the data from the two studies, preliminary estimates of the number of lives that may be saved upon completion of the Interstate System in 1972 may be developed.

Fatality Rates

	Urban	Rural
Existing highways "before" (2 studies combined)	4.1	8.1
Interstate System corridor (current study only, see table 5)	3.9	3.3
Reduction in fatality rates	0.2	4.8

The fatality rates for existing highways "before" are based on the assumption that the rates obtained in the study reported here were representative of the first one-quarter of the existing mileage programed for construction, and the fatality rates obtained in the traveled-way study were representative of the remaining mileage of existing highways. Thus, the rates for the current study were given a weight of unity; and the traveled-way study, a weight of three.

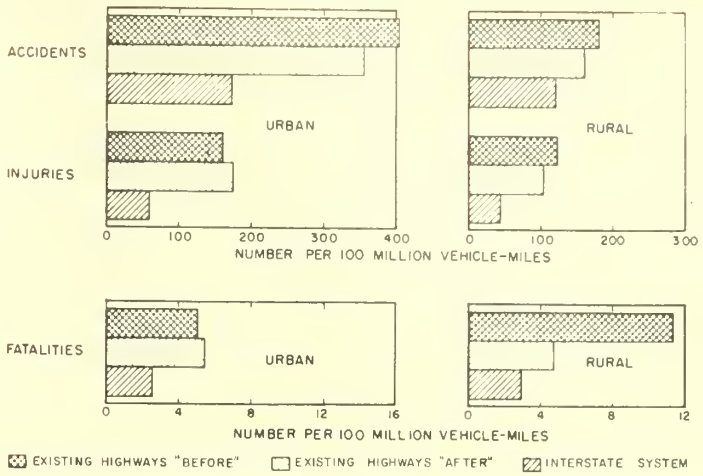

Figure 9.—Accident, injury, and fatility rates by class of highway within urban and rural areas.

Annual travel on the Interstate System at its completion in 1972 is estimated to be 240 billion vehicle-miles and total travel in the Interstate corridor is estimated at 300 billion vehicle-miles. During 1972, travel in the Interstate System corridor has been estimated, in the Highway Cost Allocation Study, at approximately 64 percent rural and 36 percent urban. In *Future Highways and*

Table 7.—Number of accidents, injuries, and fatalities and their corresponding rates by type of highway and access control

Access control by types of highways	Accidents		Injuries		Fatalities	
	Number	Number per 100 million vehicle-miles	Number	Number per 100 million vehicle-miles	Number	Number per 100 million vehicle-miles
RURAL AREAS						
Full control of access:						
Interstate highways	6,502	122	2,187	41	125	2.3
Existing highways "after"	33	140	17	72		
Existing highways "before"	5	(1)	3	(1)	1	(1)
Partial control of access:						
Interstate highways	458	91	327	65	32	6.3
Existing highways "after"	30	231	23	177	2	(1)
No control of access:						
Interstate highways	545	135	329	81	24	5.9
Existing highways "after"	2,876	160	1,914	106	84	4.7
Existing highways "before"	5,005	181	3,418	123	312	11.3
URBAN AREAS, LESS THAN 50,000 POPULATION						
Full control of access:						
Interstate highways	1,560	181	575	67	24	2.8
No control of access:						
Existing highways "after"	4,199	319	2,502	190	81	6.1
Existing highways "before"	3,189	379	1,341	160	45	5.4
URBAN AREAS, 50,000 OR MORE POPULATION						
Full control of access:						
Interstate highways	2,177	150	802	55	34	2.3
Partial control of access:						
Interstate highways	587	318	123	67	8	(1)
Existing highways "after"	279	194	106	74	3	(1)
No control of access:						
Existing highways "after"	2,464	498	810	164	23	4.6
Existing highways "before"	693	579	206	172	4	(1)

1 Too few accidents, injuries, or fatalities to be significant.

Urban Growth, it was estimated that 40 percent of the travel in the Interstate System corridor would be rural and 60 percent urban. The differences in these estimates are attributed to the method of estimating. For purposes of this study, it has been assumed that 55 percent of the Interstate System corridor travel during 1972 will be in rural areas and 45 percent in urban areas. Application of the reduction in fatalities (4.8 rural and 0.2 urban) to the estimated travel figures shows that in 1972 approximately 8,000 lives may be saved by virtue of operation of the Interstate System. It should be emphasized that the estimate of lives to be saved is based on limited data. As more information becomes available from the two continuing studies, more reliable estimates will be possible. Figure 9 illustrates accident, injury, and fatality rates in urban and rural areas for the three types of highways, based on the current analysis.

Effect of Degree of Access Control

For several years the Bureau of Public Roads has been conducting studies of the effects of access control on accidents and fatalities. Accident and fatality rates obtained for the controlled-access study and the current Interstate System accident study are shown in table 6. The results of both studies definitely demonstrate that full control of access should be used wherever possible to minimize accidents and resultant fatalities. The principal difference in comparing the data for the two studies was related to highways having partial control of access. Findings of the controlled-access study indicated that partial access control on highways in urban areas contributed little in safety benefits. Interstate System study shows that both accident and fatality rates for highways having partial access control were considerably below the rates for highways having no access control. The differences in data collected in the two studies may have been the result of the interpretation as to what constitutes partial control of access. In subsequent reports collected for this study, the effect of partial control of access will be considered in more detail.

Accident, injury, and fatality rates, distributed on the basis of extent of access control and as to rural or urban location, are shown in table 7. The benefits of modern highway design, in both rural and urban areas, are plainly evident when accident, injury, and fatility rates for highways having full control of access are compared with those having no control of access. However, for accident and injury rates, greater benefits accrue to fully controlled-access facilities located in urban areas than those in rural areas. Fatality rates on highways having full control of access in both rural and urban areas were considerably below those for highways that had no control of access; the rate decreased by approximately two-thirds in rural areas and about one-half in urban areas.

Table 8.—Number of accidents and accident rates before and after the opening of Interstate System study sections, by type of highway and average daily traffic volume

Highway	Accidents and accident rates for highways by average daily traffic volumes—						
	Under 2,000	2,000–3,900	4,000–7,900	8,000–15,900	16,000–31,900	32,000–63,900	64,000 and more
NUMBER OF ACCIDENTS							
2-lane highways:							
Existing highways "before"	122	2,242	1,929	1,439	1,065	------	------
Existing highways "after"	364	701	2,600	1,367	29	------	------
3-lane highways:							
Existing highways "before"	------	250	363	99	------	------	------
Existing highways "after"	3	395	198	28	------	------	------
4-lane undivided highways:							
Existing highways "before"	------	53	198	736	------	------	------
Existing highways "after"	19	166	781	1,159	57	------	------
4-lane divided highways:							
Existing highways "before"	------	45	120	------	------	------	------
Existing highways "after"	50	198	79	302	------	------	------
Interstate System highways	45	804	3,807	4,556	605	1,059	------
6-lane divided highways:							
Interstate System only	------	------	415	------	16	271	351
ACCIDENTS PER 100 MILLION VEHICLE-MILES							
2-lane highways:							
Existing highways "before"	157	172	228	265	475	------	------
Existing highways "after"	276	181	201	294	196	------	------
3-lane highways:							
Existing highways "before"	------	352	179	412	------	------	------
Existing highways "after"	120	188	288	566	------	------	------
4-lane undivided highways:							
Existing highways "before"	------	294	472	353	------	------	------
Existing highways "after"	316	392	354	284	241	------	------
4-lane divided highways:							
Existing highways "before"	------	157	111	------	------	------	------
Existing highways "after"	176	136	188	244	------	------	------
Interstate System highways	99	108	153	104	161	263	------
6-lane divided highways:							
Interstate System only	------	------	630	------	67	180	343

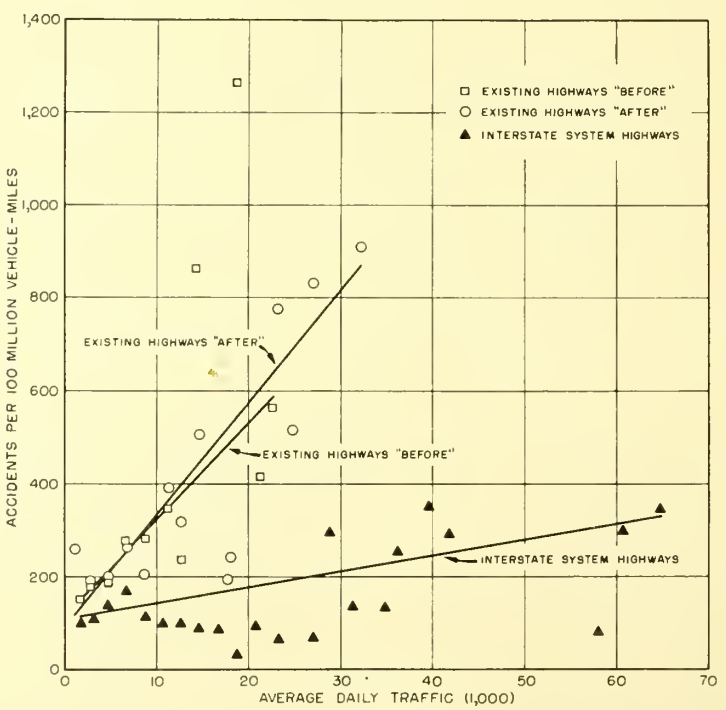

Figure 10.—Accident rates by average daily traffic volumes and class of highway study sections.

Table 9.—Comparison of accident rates, by type of collision, before and after the opening of Interstate System study sections

Type of collision or accident	Existing highways, "before"		Existing highways, "after"		Interstate System highways		Ratio (existing highways "before" ÷ Interstate System highways)
	Accidents per 100 million vehicle-miles	Percent of total	Accidents per 100 million vehicle-miles	Percent of total	Accidents per 100 million vehicle-miles	Percent of total	
Collision:							
Head-on or sideswipe, opposite direction	34	14	24	9	1	1	34.0:1
Angle	32	13	56	21	4	3	8.0:1
Collision with pedestrian	4	2	6	3	1	(1)	4.0:1
Rear-end or sideswipe, same direction	94	39	106	41	47	35	2.0:1
Other	37	16	51	19	49	36	0.8:1
Noncollision accidents	37	16	18	7	33	25	1.1:1
ALL ACCIDENTS	238	100	261	100	135	100	1.8:1

[1] Less than 0.5 percent.

Table 10.—Number of accidents occurring in rural and urban areas, by type of highway and property damage cost intervals

Property damage cost intervals	Existing highways				Interstate System only		Interstate System corridor [1]	
	"Before"		"After"					
	Number	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*
ACCIDENTS IN RURAL AREAS								
Under $100	484	9.7	367	12.5	2,041	27.2	2,408	23.1
$100–$499	2,152	42.9	944	32.1	3,432	45.7	4,376	41.9
$500 and more	1,884	37.6	505	17.2	1,962	26.2	2,467	23.6
Cost unknown	490	9.8	1,123	38.2	70	.9	1,193	11.4
TOTAL	5,010	100.0	2,939	100.0	7,505	100.0	10,444	100.0
ACCIDENTS IN URBAN AREAS								
Under $100	861	22.2	1,019	14.7	725	16.8	1,744	15.5
$100–$499	1,614	41.6	1,817	26.2	974	22.5	2,791	24.8
$500 and more	612	15.7	679	9.8	469	10.8	1,148	10.2
Cost unknown	795	20.5	3,427	49.3	2,156	49.9	5,583	49.5
TOTAL	3,882	100.0	6,942	100.0	4,324	100.0	11,266	100.0

[1] Interstate System plus existing highways "after."

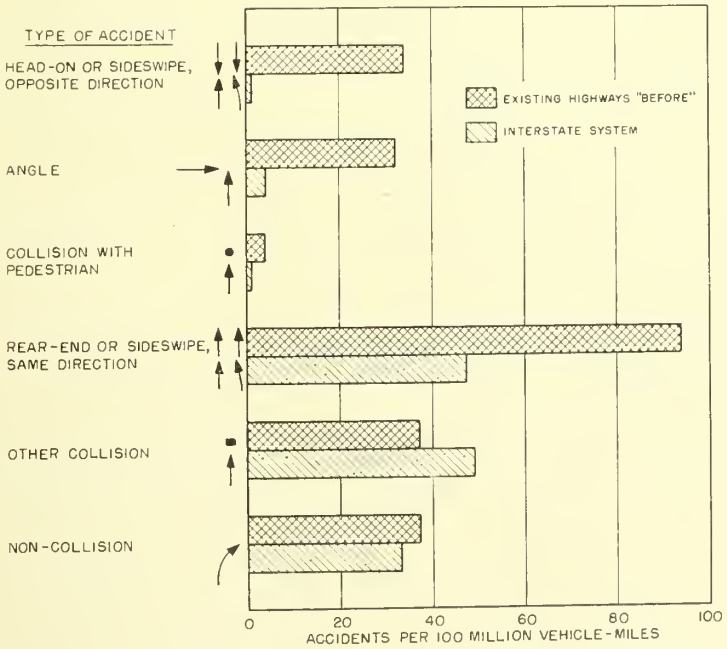

Figure 11.—Accident rates by type of accident and class of highway study section.

The extent to which access to highways is controlled is perhaps the most important design factor in terms of accident reduction. However, other features of modern design practices are also important. In rural areas, Interstate highways had lower accident and injury rates than existing highways "before" and "after," regardless of the degree of access control. This finding indicates that other design features such as wide medians, easy curvature and gradient, and long sight distances may have also contributed to the improved safety record of the Interstate System.

In initiating construction of the Interstate System, the States have concentrated their efforts in different areas. Some States have constructed their initial Interstate System mileage in congested metropolitan areas, whereas other States have built their initial mileage in rural areas. But, regardless of the type of area in which the Interstate System is constructed safety benefits accrue. The Interstate System is particularly effective in reducing the frequencies of accidents and injuries in highly urbanized areas and fatalities in rural areas.

Although type of area and control of access greatly influence incidence of accidents, the ability of the roadway to accommodate traffic or its capacity is another significant factor. Two variables closely associated with capacity are averge daily traffic volume and type of highway; that is, number of traffic lanes and whether opposing traffic is separated by a median. Other studies [3] have shown a definite relationship between average daily traffic volumes and accident rates. The study reported here confirms the relationship. Regardless of whether average daily traffic volumes are considered separately or in conjunction with types of highways, the answer is the same: traffic volumes do influence accident rates.

Tests of Independence and Linearity of Regression for regression coefficients, accident rates and ADT, were made for each of the types of highways shown in table 8. In making these tests, different average daily traffic groups were used from those shown in the table. Average daily traffic volumes in intervals of 2,000 vehicles per day were used in the tests: 0–2,000, 2,000–4,000, 4,000–6,000, and so forth. Results of the tests indicated that accident rates were dependent on average daily traffic volumes for all types of highways, except 3-lane existing "before" highways. For all highway types except 4-lane undivided existing "before" 4-lane divided existing "after," and 6-lane Interstate, the group mean accident rates followed a straightline relationship. No attempt was made to determine the de-

[3] *Effects of Average Speed and Volume on Motor-Vehicle Accidents on Two-Lane Tangents*, by D. M. Belmont, Highway Research Board Proceedings, 1953, vol. 32 p. 383–395. *The Interstate Highway Accident Study*, by M. S. Raff, PUBLIC ROADS, vol. 27, No. 8, June 1953, pp. 170–186. *The Influence of Major Highway Improvements on Traffic Accidents*, by A. H. Vey, Civil Engineering, vol. 7, No. 3, Mar. 1937, p. 213.

grees of correlation between average daily traffic volumes and accident rates for those highways whose rates did not follow a straight-line trend.

Figure 10 shows the relationship between average daily traffic and accident rates. For each of the three systems, existing "before" and "after" and Interstate, the accident rate increased as traffic volumes increased. The graph also shows that as the traffic volumes increased, Interstate safety benefits also increased, as measured by the distance between the trend lines of existing and Interstate highways. This relationship is particularly significant in that the accident rates for existing highways, as derived for the graph, included many highway types—2-lane 3-lane, and 4-lane divided and undivided. Table 4 shows the standard errors of estimate for the curves illustrated in figure 10.

Accidents by Type of Collision

Three types of collision—head-on, opposite direction sideswipe, and angle—are practically eliminated by modern design standards of the Interstate System. This finding would be expected because of the separation of opposing streams of traffic and the grade separation of intersecting highways. Table 9 and figure 11 show the frequencies of accident occurrence, by type of collision, for existing highways and Interstate highways.

Accident rates for rear-end and same direction sideswipe collisions on Interstate highways were approximately one-half those for existing highways "before" and "after." Other accident rates by type of collision, principally collision with fixed objects and noncollision accident rates, were similar for existing highways and Interstate routes.

Perhaps the most meaningful comparision of accident rates, by type of collision or accident, for Interstate highways with those for existing highways "before" is given in the extreme right column of table 9. Ratios provided therein indicate that only one type of collision happened on Interstate highways more frequently than on existing highways "before"—collisions classed as "other." The latter classification includes principally single-vehicle collisions with fixed objects. The proportion of angle collisions on the existing highways increased more than 50 percent after the Interstate System was opened to traffic. Conversely, the proportion of head-on or sideswipe (opposite direction) accidents and the proportion of noncollision accidents were reduced by approximately one-half. The diversion of through traffic to the Interstate System may have influenced these changes in accident patterns on the existing highways.

Property Damage Costs Associated With Accidents

Accident rates on the Interstate System are much lower than on other highways. Although it is not possible to make precise cost comparisons, the evidence does indicate that property damage costs for accidents on Interstate highways were no greater than those on existing highways. Table 10 summarizes property damage costs data obtained in the study. It is emphasized that data on costs other than property damage were not obtained in this study. Moreover, a high proportion of the property damage costs were reported as "unknown."

Future Reports

As more study data are collected, further analyses will be made of accidents in relation to the variables being studied: average daily traffic volumes, type of area, degree of access control, number of business or commercial establishments per mile, number of at-grade intersections per mile, type of collision, and property damage costs per accident. Such analyses will include: (1) correlations of two or more of the variables; (2) an expansion of the different highway systems' accident, injury, and fatality rate trends; (3) a more comprehensive cost analysis as accident cost study data are refined; (4) development of accident, injury, fatality, and cost equations in conjunction with future travel forecasts.

NEW PUBLICATIONS

Highway Bond Financing . . . An Analysis 1950–1962

Highway Bond Financing . . . An Analysis 1950–1962, a 45-page publication in which are reviewed the highway borrowing practices of the States, and to a lesser extent, of the local governments during the 1950–62 period, has been issued by the Bureau of Public Roads. This publication may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, at 35 cents a copy.

In this publication, highway debt is examined in terms of its magnitude, its relation to other types of debt, and its comparative costs to the highway user by means of guaranteed or revenue bond financing The effects of constitutional limitations upon creation of debt are measured and evaluated; these limitations are shown to have been largely ineffectual in restricting highway borrowing.

Some of the other facets of highway bond financing discussed include: development and impact of the authority device in financing highways by revenue bonds; comparison of interest costs and scheduled maturities of revenue bonds with other types of highway bonds; specific bond financing programs developed in selected States; resurgence in toll-road financing; and use of the authority device to finance toll-free highway programs.

The conclusion points out that the method of financing accelerated highway programs depends upon the decision to pay-as-you-go or resort to credit financing and that the use of guaranteed bonds, highway tax bonds, or short-term financing in lieu of revenue bonds can hold the costs of borrowing for highway construction to a minimum consistent with the public interest.

Estimated Travel by Motor Vehicles in 1962

BY THE CURRENT PLANNING DIVISION
BUREAU OF PUBLIC ROADS

Reported by THEODORE S. DICKERSON
Highway Engineer

MOTOR-VEHICLE travel in the United States in 1962 totaled 767.8 billion vehicle-miles, an increase of 4.1 percent over the travel in 1961. The travel data were compiled from information supplied by the State highway departments and toll authorities. Total travel for 1963, based on information for the first 9 months of the year is estimated at 798 billion vehicle-miles, a 4-percent increase over 1962.

The proportions of travel by road system and by vehicle type changed little from 1961 to 1962. Of the 1962 travel, 40 percent was on main rural roads comprising 14 percent of the Nation's total of 3.6 million miles of roads and streets. Another 46 percent of the travel was on urban streets, which comprise only 13 percent of the total mileage. Local rural roads accounted for only 14 percent of the travel but make up 73 percent of the total mileage.

Passenger cars represented 83.5 percent of the vehicles registered and accounted for 81.8 percent of the travel in 1962; trucks and truck combinations accounted for 16.1 percent of the vehicles registered and 17.6 percent of the travel. Buses accounted for 0.4 percent of all vehicles registered and for 0.6 percent of total travel.

Average vehicle performance in 1962 differed very little from that reported for 1961. The average motor vehicle traveled 9,635 miles in 1962, almost half of it in cities, and consumed 774 gallons of fuel at a rate of 12.44 miles per gallon. The average passenger car traveled 9,435 miles and consumed 654 gallons of fuel at a rate of 14.42 miles per gallon. The average truck traveled a little more and the average commercial bus a little less in 1962 than in 1961, but their average rates of fuel consumption did not change appreciably.

The travel and related information for 1962 is shown in table 1 by road system and vehicle type. Such data have been reported in PUBLIC ROADS magazine for a number of years; the latest, for 1961, appeared in vol. 32, No. 7, April 1963, p. 180.

Table 1.—Estimated motor-vehicle travel in the United States and related data for calendar year 1962 [1]

Vehicle type	Motor-vehicle travel					Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural road travel	Local rural road travel	Total rural travel	Urban travel	Total travel			Total	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles/gallon*
Passenger cars [2]	242,521	82,099	324,620	303,619	628,239	66,589	9,435	43,570	654	14.42
Buses:										
Commercial	915	165	1,080	1,776	2,856	75.5	37,828	609	8,066	4.69
School and nonrevenue	656	694	1,350	270	1,620	209.7	7,725	229	1,092	7.07
ALL BUSES	1,571	859	2,430	2,046	4,476	285.2	15,694	838	2,938	5.34
All passenger vehicles	244,092	82,958	327,050	305,665	632,715	66,874	9,461	44,408	664	14.25
Trucks and combinations	66,092	21,460	87,552	47,507	135,059	12,809	10,544	17,288	1,350	7.81
ALL MOTOR VEHICLES	310,184	104,418	414,602	353,172	767,774	79,683	9,635	61,696	774	12.44

[1] For the 50 States and District of Columbia.
[2] Includes taxicabs; also motorcycles (660,400 registered).

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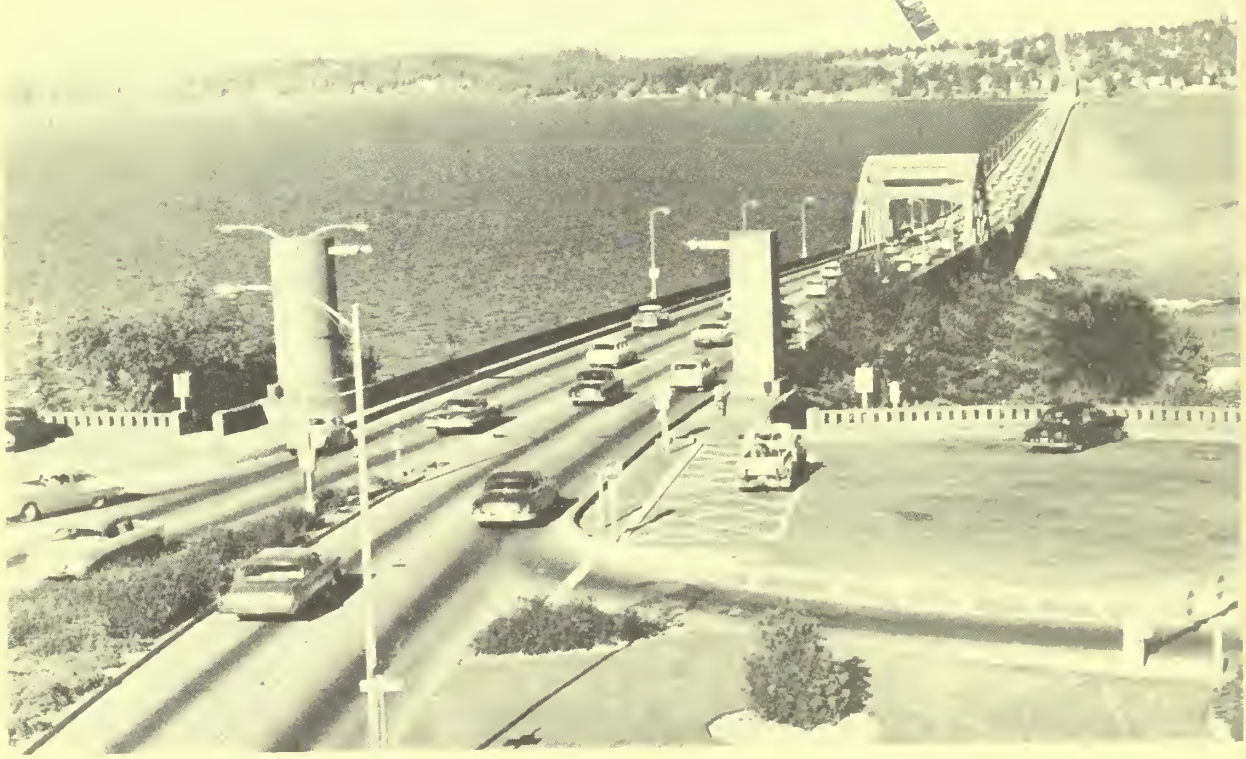

Lake Washington Floating Bridge on Interstate Route 90–1, major entry into Seattle, Washington from the east.

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U.S. DEPARTMENT OF COMMERCE
LUTHER H. HODGES, Secretary
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Dimensions and Weights of Highway Trailer Combinations and Trucks, 1959

BY THE TRAFFIC SYSTEMS RESEARCH DIVISION BUREAU OF PUBLIC ROADS

Reported by[1] **MALCOLM F. KENT,** Transportation Economist, and **HOY STEVENS,** Highway Transport Research Engineer

Small 2-axle, single-unit trucks are the predominant freight vehicle in use on all but Interstate and main rural primary roads; however, on Interstate and main rural primary roads the intercity, line-haul freight is generally hauled in trailer combinations. It is for this use that large trailer combinations are an important part of the Nation's transportation system. Many communities are served only by trailer combinations for their incoming and outgoing freight deliveries. Because of the importance and amount of line-haul freight transport by highway commercial vehicles, it is of value to highway planners as well as to highway engineers to have data on the types, sizes, gross weights, degree of loading, and numbers of trailer combinations and trucks actually in use.

This article provides information showing the range and distribution of freight vehicles in use in 1959 in terms of length, width, height, and weight. Since 1959, large increases have occurred in use of trailer combinations, such as the use on Michigan's rural primary roads of 10- and 11-axle trailer combinations for which the loaded gross weights were more than 140,000 pounds, within the overall length limitation of 55 feet. Also, 9-axle trailer combinations having overall lengths of approximately 100 feet and carrying gross weights of 128,000 to 130,000 pounds are now being permitted on some toll roads. By 1963 the predominant length of new semitrailers and full trailers in use had increased to 40 feet, 5 feet more than the 35-foot predominant length in 1959.

Periodic sampling of dimensions and gross weights of trailer combinations currently in use on urban and rural highways is needed to keep the highway planning and design engineers informed as to the sizes and weights of highway freight vehicles being permitted and used on the highways. This type of information on trailer combinations and trucks is also useful in economic and public finance studies and for making estimates on the cargo-carrying potential of highway freight transport vehicles available for emergency situations and National Defense.

Introduction

A SAMPLING of the weights of highway freight trailer combinations and single-unit trucks is obtained by the highway departments in most of the States each year. But, no precise census of the number and type of trailers in highway freight service is available in the United States because of the multiple registration of trailers in more than one State, the short trailers used principally in city service, and the trailers used only for utility and construction purposes.

Because new cargo bodies of 40 feet and longer are entering the traffic stream, a need exists for repeating the size and dimension study periodically so that data on cargo-carrying capabilities of highway vehicles may be kept current. Therefore, in 1959, the Bureau of Public Roads collected data regarding the dimensions and weights of 155,300 vehicles—both empty and loaded—of which 90,200 were trailer combinations and 65,100 were single-unit trucks. A study and analysis of these data are presented in this article.[2] It is possible that some vehicles and combinations may have been weighed and measured more than once because of the location of the weighing stations, the period of time for which the stations were used at a specific location, and the random selection of vehicles and combinations in transit past the station. However, the sample is believed to represent a cross section of the automotive freight vehicles in use in the continental United States in 1959. Insofar as trailer combinations are concerned, the data portray the trucking industry's use of sizes and weights under the legal limitations in effect for several years prior to 1959.

Information was published in 1958 from a study on the demand for highway transportation (*1*)[3] in terms of shipping densities of commodities and tons involved in the five principal media of transportation. In 1957, according to an estimate printed in 1961 (*2*), there were 712,129 semitrailers and full trailers in highway freight service in the United States being used in 602,457 trailer combinations for rural and intercity highway freight transportation. This article includes data that show a distribution of highway freight vehicles by weights and dimensions in 1959.

By the nature of the trucking industry, vehicle sizes and cubic capacities are not immediately changed to take advantage of permitted increases in sizes, although any additional weight allowances are used in the hauling of the heavier commodities in the currently owned vehicles. Older and smaller vehicles usually are operated until no longer serviceable, although a pressure develops for their earlier retirement and replacement when legal limitations are raised to higher levels. Changes in legal limitations are dependent upon technological developments of vehicles, upon changes in the characteristics and amount of highway freight transport, and upon improvement in the design and construction of a State's highway system. The amount of highway freight has been increasing during the past several years, and this increase has caused the motor carriers to press for larger, more efficient vehicles.

[1] Presented at the 42d annual meetiug of the Highway Researeh Board, Washington, D.C., January 1963.

[2] Instructions and proce lures for obtaining the data as part of the 1959 truck weights study were developed by Alexander French, Chief of the Planning Services Branch, Office of Planning. Miss Mildred M. Milazzo, Mrs. Madalene H. Kendall, and Mrs. Kathleen V. Toole of the Vehicle Research Branch helped to arrange the field data, prepare the data for machine analysis, and develop the summary tables and charts. John H. Jones of the Data Processing Division, Office of Administration, made the machine tabulations and suggested forms for the tables, which were produced by the electrie accounting machine.

[3] Figures in parentheses indicate references listed on page 285.

During recent years, legal limitations have been revised to afford additional transport efficiency to motor carriers. The extent of these changes over the 5-year period, May 1957 to July 1962, are evident from the data on increases in lengths allowed for semitrailers and are shown in the appendix. The length limitations in effect in 1957 and previous years probably had a controlling influence on the lengths of semitrailers recorded in the 1959 weight studies. In 1957, 31 States permitted semitrailers 40 feet or longer, although 18 States prohibited the use of 40-foot semitrailers in long haul interstate service. By 1962, this prohibition had been eased and only West Virginia (35 feet) and Georgia (39.5 feet) restricted trailer length to less than 40 feet. For States having no statutory limit on semitrailer length, the maximum possible semitrailer length was assumed to be 7 feet less than the permitted length of the tractor semitrailer combination. This 7-foot dimension consists of a bumper-to-rear-of-cab dimension of 4 feet, obtainable for cab-over-engine tractors, plus 3 feet of clearance between rear of cab and nose of semitrailer.

Definitions

Trailer combinations

Trailer combinations are classified according to the axle classification code developed by the Bureau of Public Roads. In this code, each digit represents the number of axles of one vehicle in the combination. The symbol for a trailer combination consists of two or three digits separated by hyphens. The first digit represents the power vehicle, either a truck tractor or a tractive truck (a truck equipped to carry a cargo body and haul a full trailer). An "S" before the second digit indicates a semitrailer, the power vehicle being a tractor. A digit appearing without an "S" in either the second or third position in a combination symbol indicates a full trailer. Some examples are given in the following sentences.

The code for a 3-axle tractor and a 2-axle semitrailer combination is 3–S2. Codes for double cargo body combinations include: 3–2 for a 3-axle tractive truck and a 2-axle full trailer; 2–S1–2 for a 2-axle tractor plus a 1-axle semitrailer and a 2-axle full trailer. Such combinations are also known as double trailer combinations.

Cargo body types

Approximately 40 types of cargo bodies are defined in the Society of Automotive Engineers publication, *Commercial Motor Vehicle Nomenclature* (*3*). However, such a detailed classification was not suitable for use in this study and descriptive terms were used to group cargo bodies according to their similarity in cargo containing characteristics. The following list shows the types of vehicles included in each classification used in this study.

Flatbed: Platform (flat or stake), low-bed, riggers or oil field, lumber, and express or pickup bodies.

Van: Livestock rack, canopy, open-top box, fully enclosed van, insulated van, furniture or moving van, bottler, multistop or standup delivery, and panel truck bodies.

Log: Log, pulpwood, or pipe bodies.

Dump: Grain, dump low side open box, and hopper bodies.

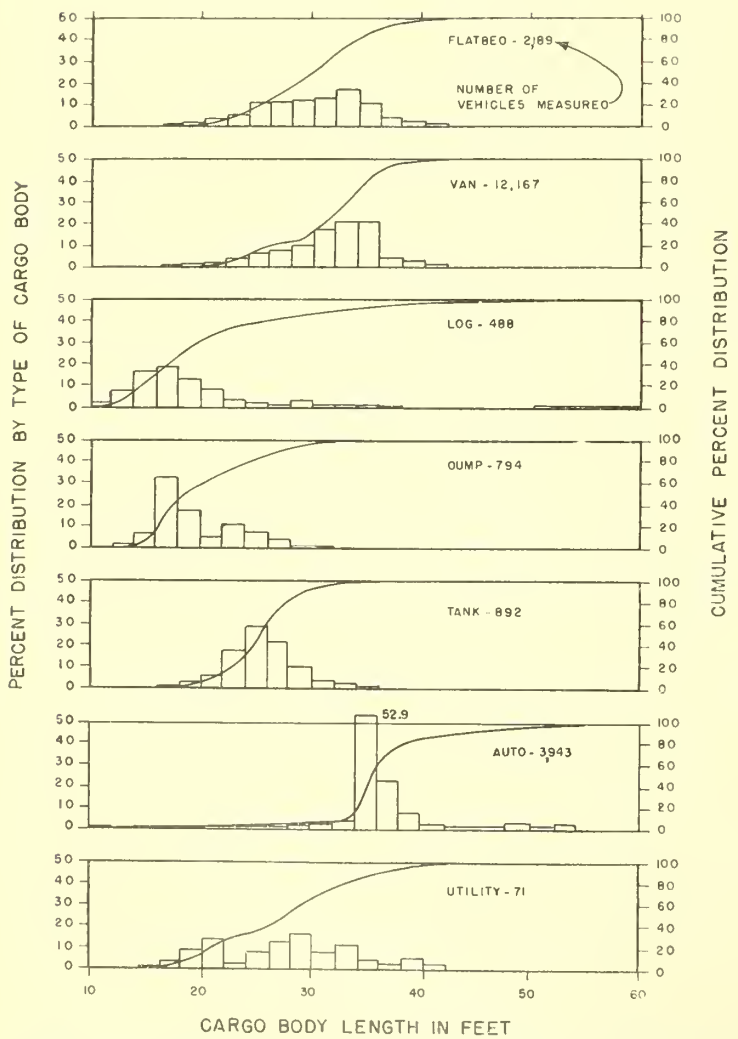

Figure 1.—Distribution of cargo body lengths, 3-axle trailer combinations.

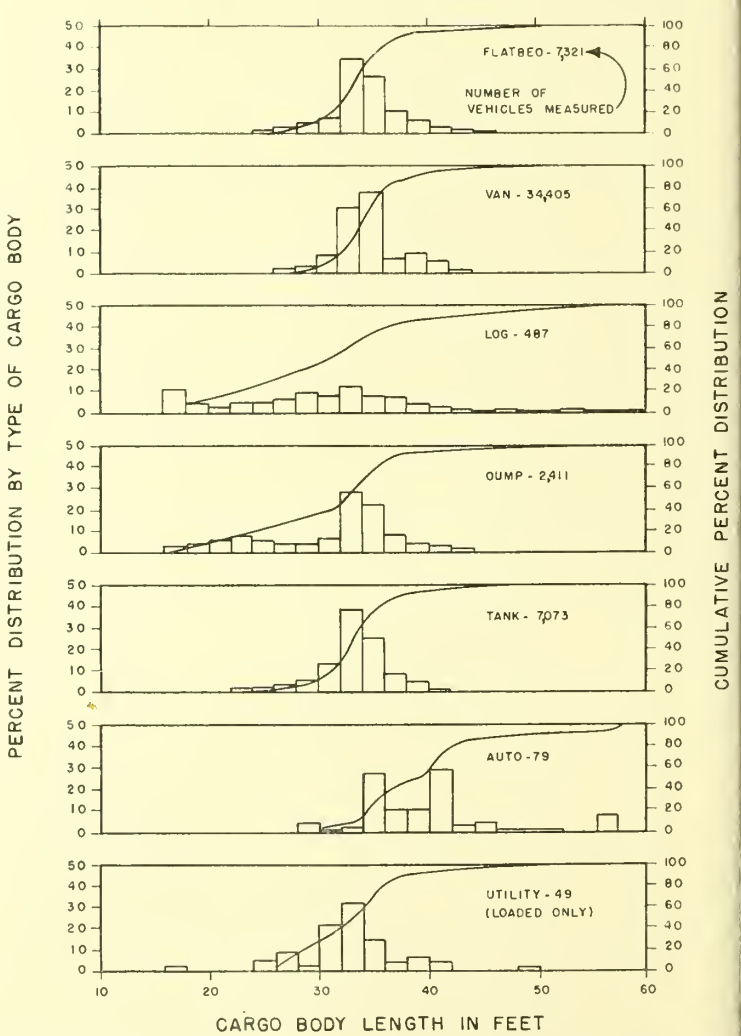

Figure 2.—Distribution of cargo body lengths, 4-axle trailer combinations.

Tank: Petroleum, insulated and uninsulated; bituminous distributor; and other liquid product bodies (milk, acids, sugar, etc.).

Auto: Bodies designed primarily for transportation of other vehicles.

Concrete: Bodies designed and equipped to mix and agitate concrete.

Utility: Wrecker; utility (transportation of tools; equipment; and supplies for construction, maintenance and repair purposes); garbage, refuse, lift and equipment (tank-mounted cranes, well drills, compressors, etc.) bodies.

Empty vehicle weight

The empty weight of a vehicle or trailer combination is its weight with fuel and without cargo or payload; it may include fixtures permanently carried to support the payload.

Loaded gross weight

The loaded gross weight of a trailer combination is its empty weight plus the weight of the cargo or payload carried.

Summary

Some of the more important findings from the 1959 study are stated in the following paragraphs.

Most of the van and flatbed cargo bodies of semitrailer combinations were 35 feet long in 1959. Because 40-foot cargo bodies have been constructed since 1959 in significant numbers, periodic studies of cargo body lengths will be necessary to provide current information on highway freight movement usage and capabilities.

Van cargo bodies on 2-axle, 6-tired trucks averaged about 12 to 14 feet in length and van cargo bodies on 3-axle trucks averaged about 18 to 20 feet.

Empty weights of 3–S2, 2–S1–2, and 3–2 trailer combinations averaged about 30,000 pounds; and empty weights of 2–S1 and 2–S2 combinations averaged about 20,000 and 25,000 pounds, respectively. Average empty vehicle weights of five different vehicle classes, all having van bodies, increased in step intervals of approximately 5,000 pounds, as follows: 2-axle, 6-tired trucks, 9,300 pounds; 3-axle trucks, 15,200 pounds; 2–S1 combinations, 20,100 pounds; 2–S2 combinations, 24,800 pounds; and 3–S2 combinations, 30,700 pounds.

Average empty weights of van body, single-unit trucks were: panels, 6,100 pounds; other 2-axle, 4-tired trucks, 6,400 pounds; 2-axle, 6-tired trucks, 9,300 pounds; and 3-axle trucks, 15,200 pounds. The 2–S2 combinations on the average had loaded gross weights of about 15,000 pounds more than those of the 2–S1 combinations of the same body types. The loaded gross weights of 3–S2 combinations having flatbed, van, and tank cargo bodies were about 11,000 pounds more on the average than those for the 2–S2 combinations having these same body types. The loaded gross weights of 2–S1–2 combinations were between 28,000 and 38,000 pounds more than those for the 2–S1 combinations in the States where the double cargo combinations are permitted.

The average payload of 10,800 pounds carried by 2–S1 van combinations was 12,000 pounds less than the payload carried by 2–S2 van combinations and 16,000 pounds less than that carried by 3–S2 van combinations.

The number of trailer combinations having loaded gross weights of more than 60,000 pounds was, as follows: 35 percent of the total number of the loaded combinations weighed in the States having a maximum gross weight limit of 60,000 pounds; 41 percent in the States having a maximum gross weight limit of 65,000 pounds; 50 percent in the States having a maximum gross weight limit of 76,000 pounds; and nearly 64 percent in those States having maximum gross weight limits of 78,000 pounds or more.

Approximately 10 percent of the total of the 2-axle, 6-tired trucks, the 3-axle trucks, and the trailer combinations exceeded the 8-foot width limitation, and only about 1 percent of the total exceeded the width limitation of 8.3 feet. Approximately 0.3 percent of all vehicles were more than 13 feet 6 inches high. A greater percentage of the 3–S2 combinations exceeded this height than any of the other classes of vehicles.

Procedures

Loaded and empty weights and the dimensions of highway cargo vehicles were obtained at truck weighing stations in 46 States during 1959. These vehicles were classified as to axle arrangement and type of cargo body. The lengths of cargo bodies were arrayed in 2-foot intervals, which provided a means of investigating the cubic capacities of cargo bodies in use during 1959. The greatest number of cargo bodies had lengths of 32 to 36 feet.

Loaded and empty weights of vehicles were averaged and the resultant averages were used to compute average payloads by type and length of cargo body for each vehicle classification. Average empty weights of five different vehicle classes of vehicles having van bodies increased in step intervals of approximately 5,000 pounds. These five average empty weights were: 2-axle, dual-rear-tire truck, 9,300 pounds; 3-axle truck, 15,200 pounds; 2–S1 trailer combination, 20,100 pounds; 2–S2 trailer combination, 24,800 pounds; and the 3–S2 trailer combination, 30,700 pounds. The 4-axle (2–S2), tractor van-semitrailer combination carried on the average about 6 tons more payload than the 3-axle (2–S1), tractor van-semitrailer combination, and 2 tons less than the 5-axle (3–S2), tractor van-semitrailer combination.

In the States that limited gross weights of motor vehicles to 56,000 to 60,000 pounds, the greatest percentage of loaded gross weights was in the 50,000- to 60,000-pound weight bracket. But weights recorded for States that have higher gross weight limits had the greatest percentage of loaded gross weights in the 60,000- to 70,000-pound weight bracket. About 1 percent of all trailer combinations and 1 percent of all 2- and 3-axle, dual-tired trucks exceeded 8.3 feet in width across the wheels and approximately 0.3 percent of the vehicles exceeded 13.5 feet in height.

Length of Cargo Bodies in the Traffic Stream

In 1959, the length, height, and width of 155,300 commercial cargo vehicles were recorded as the vehicles were weighed at truck weighing stations in 46 States (the District of Columbia was included and treated as a State). Although at least one estimation (4) had been made by the motor-vehicle industry of the lengths of van trailers by year of construction, no industry tabulation had been made available that would give a cross section of cargo motor vehicles operating on the highways at any given time. Dimensional information concerning new vehicles going into the traffic stream each year would be useful, but a cross section of the vehicles, old and new, on the highways would give a better understanding of highway freight movement capabilities.

Trailer combinations

In figure 1 the percentage distributions of cargo body lengths and the cumulative percentage curves of 2–S1 trailer combinations are shown. The 20,544 sample of 2–S1 combinations included flatbed, van, auto, log, dump, tank, and utility bodies. Fifty percent of the flatbed bodies were more than 30 feet long, and 50 percent of the van bodies were more than 32 feet long. Most of the dump bodies were 16 to 18 feet long, and most of the tanks were 24 to 26 feet long. Automobile carrier bodies were predominantly 34 to 36 feet long and log bodies were mostly 16 to 18 feet long. Utility body lengths were spread rather evenly over the wide range of 14 to 42 feet.

A rather marked difference in the configurations of the distributions of cargo body lengths in 2–S2 combinations can be seen in figure 2 as compared to the distributions of cargo bodies in 2–S1 combinations. The distributions of cargo body lengths in 2–S2 combinations show a predominance of 32- to 34-foot lengths for all but vans and auto carriers, which were mostly 34 to 36 feet long. The data shown here are for 1959, before the advent of 40-foot cargo bodies in appreciable numbers.

Because 40-foot trailers have been built in considerable numbers since 1959 it would seem advisable to repeat this vehicle dimension study every 3 to 5 years to study the extent of the addition of longer cargo bodies to the traffic stream. Periodic studies would also reflect what lengths of cargo bodies were being retired from service. In this connection, note that of the 34,405 van cargo bodies measured on 2–S2 combinations approximately 36 percent were 34 to 36 feet long and 30 percent were 32 to 34 feet long. Thus in 1959 two-thirds of the 2–S2 van cargo bodies were 32 to 36 feet long and 42 and 47 percent respectively of the 2–S1 and 3–S2 van cargo bodies were 32 to 36 feet long.

The 3–S2 tractor semitrailer combinations had a marked predominance of 35-foot cargo bodies, except for the log and utility body types, as shown in figure 3. Fifty-eight percent of the 3–S2 tanks were at least 36 feet long and some were longer, but only about 14 per-

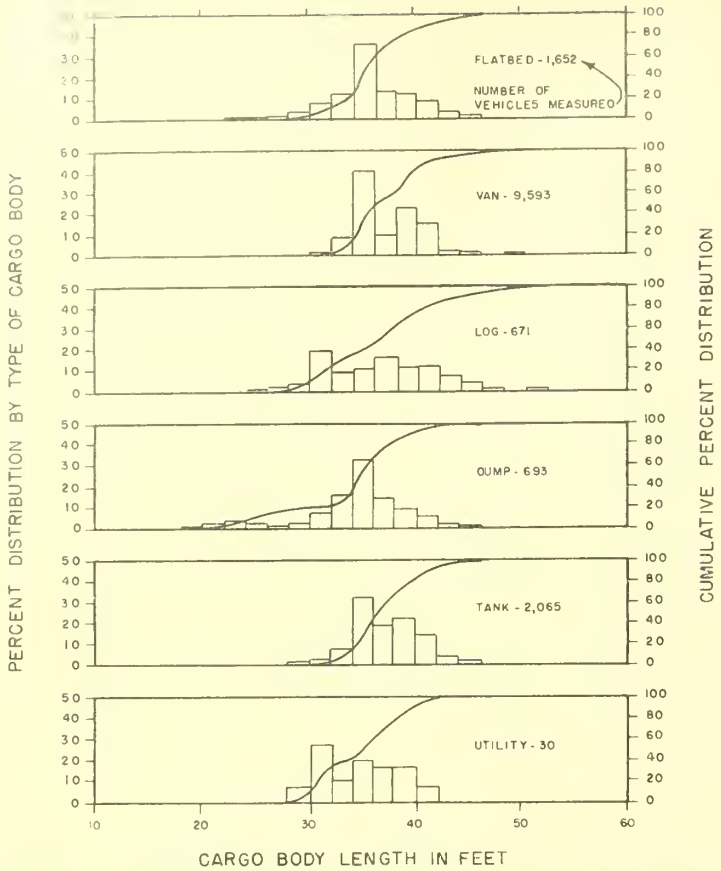

Figure 3.—Distribution of cargo body lengths, 5-axle tractor trailer combinations.

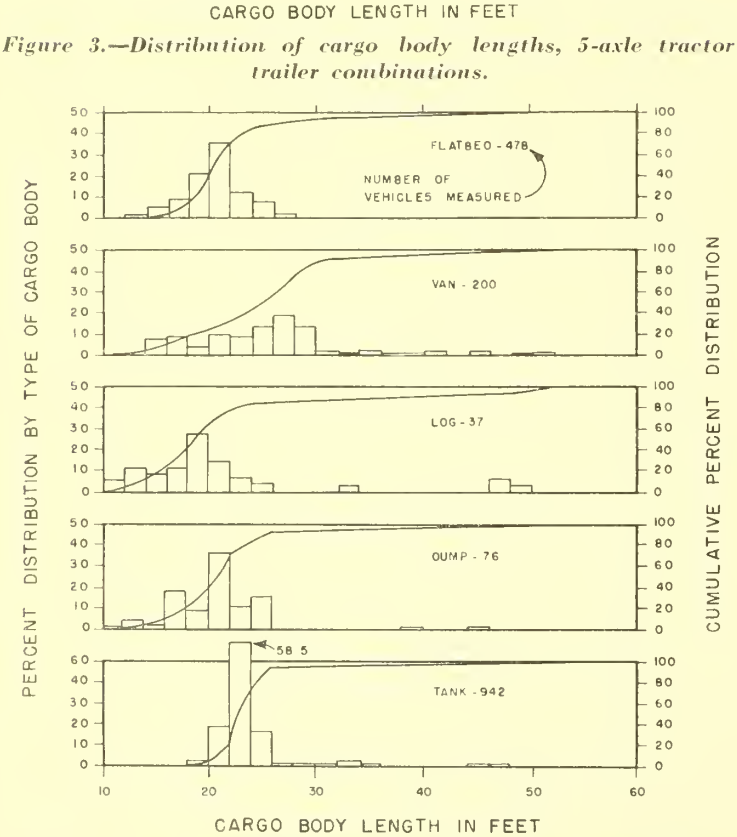

Figure 4.—Distribution of cargo body lengths, tractive-truck full-trailer combinations.

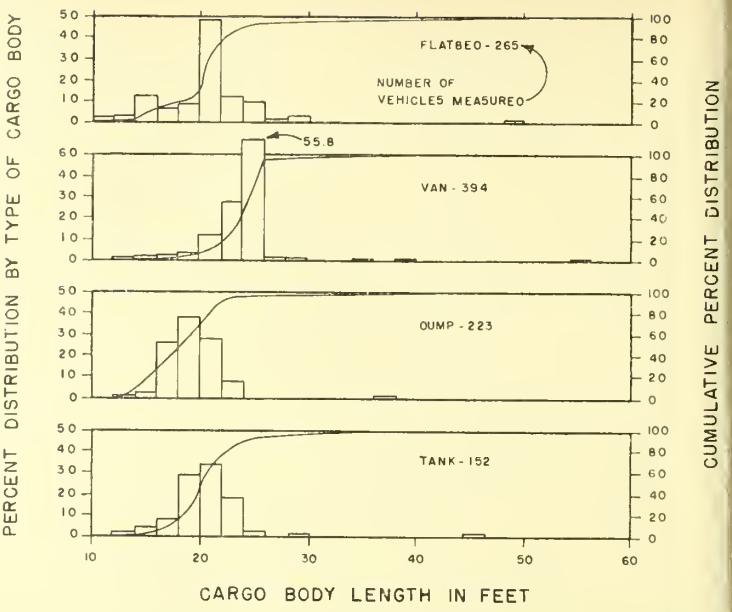

Figure 5.—Distribution of cargo body lengths, tractor semitrailer full-trailer combinations.

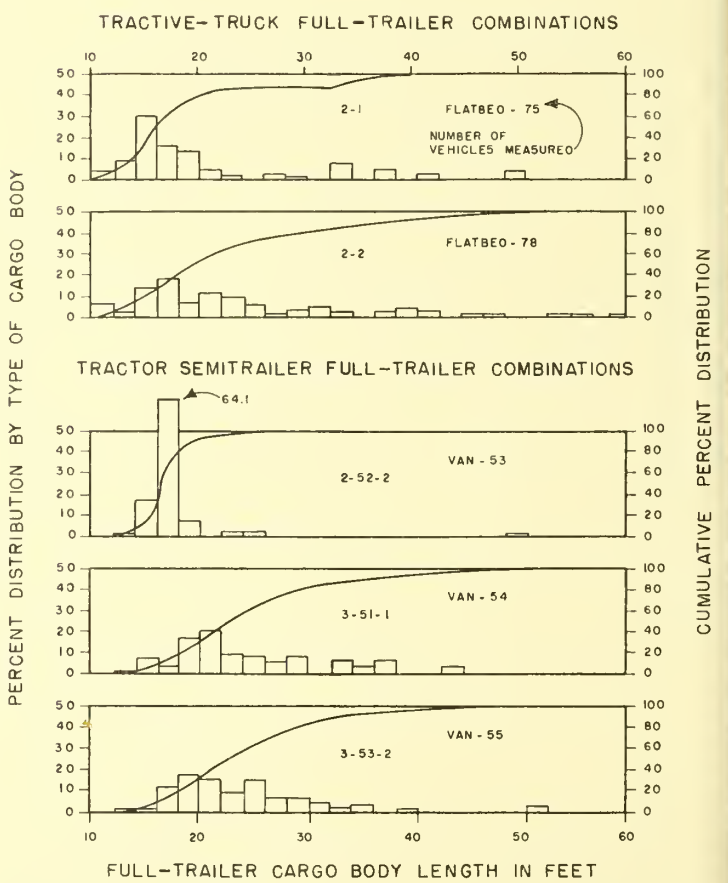

Figure 6.—Distribution of cargo body lengths, miscellaneous vehicles.

cent of the 2-S2 tanks and 1 percent of the 2-S1 tanks were 36 feet long. Length of cargo bodies for log trailers ranged evenly from 30 to 46 feet. Only 30, 5-axle tractor-utility-trailers were counted in this study and their cargo bodies were from 28 to 42 feet long.

Although high percentages of the total double cargo body combinations counted were weighed and measured, the samples were small in number as compared to the samples obtained for single cargo body combinations. The data for the 3-2, tractive-truck, full-trailer combinations are reported in figure 4, and the data for the 2-S1-2 tractor, semitrailer, full-trailer combinations are shown in figure 5. The lengths of the semitrailers in the 2-S1-2 combinations were the same as the lengths of the full-trailers in such combinations.

Most of the flatbed, full-trailers used in the 3–2 and 2–S1–2 trailer combinations were 20 feet long. Ninety percent of the van, full-trailers in the 3–2 combinations were less than 30 feet long, and 97 percent of the van, full-trailers used in the 2–S1–2 combinations were less than 26 feet long. Dump, full-trailers were mostly 16 to 22 feet long. Tank, full-trailers in 3–2 combinations were mostly 22 to 24 feet long, and lengths of the tank trailers in the 2–S1–2 combinations were rather evenly distributed from 18 to 24 feet. The basic data used for figs. 1–5 are given in tables 1 and 2.

In figure 6, trailer length distributions of five different, double cargo body trailer combinations are shown for between 50 and 100 observations made for each combination. The 2–1 class of trailer combinations has a limited local use, usually as a seasonal, auxiliary freight vehicle in agricultural areas. The trailers observed in such combinations were flatbed, balanced full-trailers and were from 14 to 20 feet long. In the 2–2 class of trailer combinations, only flatbed full-trailers were observed and they had a predominant range in length from 14 to 26 feet. This class of 2–2 trailer combinations apparently was not adequately sampled in this study because other types of bodies are used in this class of trailer combinations.

Data collected for tractor, semitrailer, and full-trailer combinations (2–S2–2, 3–S1–1, and 3–S3–2), three classes of trailer combinations less frequently used than others, also are shown in figure 6. All of these had van full-trailers. The 2–S2–2 van trailers were mostly 16 to 18 feet long; the lengths of van trailers in both 3-axle tractor combinations were spread over a wide range from 14 to 40 feet, most of them were from 18 to 26 feet long.

Cumulative percentage curves for all the 2–S1, 2–S2, 3–S2, 3–2 and 2–S1–2 combinations, by cargo body type, are shown in figure 7 for easy comparison. Of course, some long cargo bodies reported may have been special permit vehicles. These charts indicate that in 1959 no predominant length of trailer was used for all purposes, the predominant lengths ranged from 20 to 40 feet.

Trucks

Of the 268 pickup trucks, 85 percent had cargo bodies 6 to 10 feet long and the 614 panel trucks were evenly distributed as to length over the 2-foot intervals from 6 to 18 feet, figure 8 and table 3. Few panel and pickup trucks were recorded on rural roads; their primary use is in urban areas.

Two-axle motortrucks having 4 tires, other than panels and pickups, had cargo body measurements similar to the panels and pickups; most of the flatbed bodies were 6 to 10 feet long and the length of van bodies ranged from 6 to 20 feet. Two-axle motortrucks having 6 tires had cargo body lengths mostly in the range of 12 to 16 feet, except for dump trucks and utility vehicles. Dump bodies had average lengths of about 10 feet and utility body lengths were rather evenly distributed over the range of 8 to 16 feet, figure 9 and tables 3 and 4.

Table 1.—Distribution of trailer body lengths in combinations, 46 States

Trailer body length	2-S1		2-S2		3-S2		3-2		2-S1-2	
FLATBED										
Feet	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*
10–11.9	4	0.2								
12–13.9	11	0.5							4	1.5
14–15.9	7	0.3							29	10.9
16–17.9	32	1.5	12	0.2	3	0.2	80	16.7	17	6.4
18–19.9	40	1.8	4	0.1	1	0.1	104	21.8	22	8.2
20–21.9	105	4.8	16	0.2	3	0.2	168	35.2	130	48.7
22–23.9	118	5.4	33	0.5	9	0.5	55	11.5	31	11.6
24–25.9	251	11.5	80	1.1	10	0.6	35	7.3	23	9.0
26–27.9	269	12.3	168	2.3	21	1.3	20	4.2	3	1.1
28–29.9	278	12.7	433	5.9	57	3.5	7	1.5	4	1.5
30–31.9	294	13.4	875	12.0	113	6.8	2	0.4		
32–33.9	367	16.7	2,534	34.5	202	12.2	1	0.2		
34–35.9	223	10.2	1,843	25.1	578	35.0				
36–37.9	101	4.6	655	8.9	219	13.3				
38–39.9	55	2.5	373	5.1	205	12.4				
40–41.9	16	0.7	181	2.5	137	8.3				
42–43.9	9	0.4	45	0.6	38	2.3				
44–45.9	1	0.1	34	0.5	31	1.9	1	0.2		
46–47.9			11	0.2	5	0.3				
48–49.9	5	0.2	10	0.1	7	0.4	2	0.4		0.4
50–52 and over	3	0.2	14	0.2	13	0.7	3	0.6	2	0.7
Total	2,189	100.0	7,321	100.0	1,652	100.0	478	100.0	265	100.0
VAN										
10–11.9	8	0.1								
12–13.9	9	0.1							1	0.2
14–15.9	15	0.1							3	0.8
16–17.9	69	0.6	21	0.1	12	0.1	38	19.0	6	1.5
18–19.9	83	0.7	14	0.1	10	0.1	9	4.5	8	2.0
20–21.9	314	2.6	38	0.1	12	0.1	19	9.5	44	11.2
22–23.9	490	4.0	91	0.3	10	0.1	19	9.5	100	25.3
24–25.9	774	6.4	117	0.3	21	0.2	29	14.5	220	55.8
26–27.9	947	7.8	199	0.6	13	0.1	38	19.0	5	1.3
28–29.9	1,200	9.9	799	2.3	36	0.4	27	13.5	3	0.8
30–31.9	2,011	16.5	3,212	9.3	134	1.4	2	1.0		
32–33.9	2,561	20.9	10,329	30.0	744	7.8	1	0.5		
34–35.9	2,543	20.9	12,386	35.9	3,786	39.5	3	1.5	2	0.5
36–37.9	518	4.3	2,233	6.5	793	8.3	1	0.5		
38–39.9	416	3.4	2,941	8.5	2,129	22.2	1	0.5	1	0.3
40–41.9	158	1.3	1,648	4.8	1,386	14.4	4	2.0		
42–43.9	28	0.2	259	0.8	230	2.4				
44–45.9	13	0.1	68	0.2	126	1.3	2	1.0		
46–47.9	7	0.1	19	0.1	42	0.4				
48–49.9			23	0.1	46	0.5	1	0.5		
50–52 and over	3	0.0	8	0.0	63	0.7	6	3.0	1	0.3
Total	12,167	100.0	34,405	100.0	9,593	100.0	200	100.0	394	100.0
LOG										
10–11.9	13	2.7								
12–13.9	42	8.6							NA	NA
14–15.9	84	17.2							NA	NA
16–17.9	96	19.7	55	11.3	2	0.3	13	35.2	NA	NA
18–19.9	67	13.7	21	41.3	1	0.1	10	27.0	NA	NA
20–21.9	45	9.2	19	3.9	2	0.3	5	13.5	NA	NA
22–23.9	21	4.3	27	5.6	1	0.1	2	5.4	NA	NA
24–25.9	20	4.1	28	5.8			1	2.7	NA	NA
26–27.9	13	2.7	34	7.0	13	2.0			NA	NA
28–29.9	9	1.8	44	9.0	32	4.8			NA	NA
30–31.9	17	3.5	40	8.2	128	19.0			NA	NA
32–33.9	10	2.1	70	14.4	59	8.8	1	2.7	NA	NA
34–35.9	10	2.1	44	9.0	80	12.0			NA	NA
36–37.9	11	2.3	34	7.0	103	15.3			NA	NA
38–39.9	9	1.8	21	4.3	72	10.7			NA	NA
40–41.9	6	1.2	15	3.1	76	11.3			NA	NA
42–43.9	3	0.6	8	1.6	48	7.1			NA	NA
44–45.9	2	0.4	4	0.8	30	4.8			NA	NA
46–47.9	2	0.4	6	1.2	9	1.3	2	5.4	NA	NA
48–49.9	2	0.4	4	0.8	3	0.4	1	2.7	NA	NA
50–52 and over	6	1.2	13	2.7	12	1.7	2	5.4	NA	NA
Total	488	100.0	487	100.0	671	100.0	37	100.0	NA	NA
DUMP										
10–11.9	1	0.1								
12–13.9	8	1.0							1	0.4
14–15.9	58	7.3							3	1.4
16–17.9	267	33.6	76	3.2			20	26.3	57	25.6
18–19.9	133	16.8	88	3.6	8	1.2	7	9.2	85	38.1
20–21.9	69	8.7	141	5.8	20	2.9	27	35.6	59	26.5
22–23.9	39	4.9	155	6.4	26	3.8	8	10.5	17	7.6
24–25.9	90	11.3	120	5.0	20	2.9	12	15.8		
26–27.9	57	7.2	100	4.1	14	2.0				
28–29.9	31	3.9	101	4.2	19	2.7				
30–31.9	16	2.0	147	6.1	43	6.2				
32–33.9	12	1.5	659	27.3	104	15.0				
34–35.9	3	0.4	496	20.6	221	31.9				
36–37.9	7	0.9	173	7.2	95	13.7			1	0.4
38–39.9	2	0.3	82	3.4	63	9.0	1	1.3		
40–41.9	1	0.1	40	1.7	36	5.2				
42–43.9			21	0.9	13	1.9				
44–52 and over			12	0.5	11	1.6	1	1.3		
Total	794	100.0	2,411	100.0	693	100.0	76	100.0	223	100.0

Table 2.—Distribution of cargo body lengths in trailer combinations, 46 States

Cargo body length	2-S1		2-S2		3-S2		3-2		2-S1-2		2-S1		2-S2		2-S1		2-S2		3-S2	
	TANK										AUTO				UTILITY					
Feet	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*
10-11.9											4	0.1								
12-13.9	1	0.1							3	2.0	1	0.0								
14-15.9	2	0.2							7	4.5	2	0.1			1	1.4				
16-17.9	11	1.2	9	0.1	2	0.1	14	1.5	13	8.5	3	0.1			2	2.8	1	2.0		
18-19.9	32	3.6	3	0.1			26	2.7	45	29.4	2	0.1			6	8.4				
20-21.9	54	6.1	13	0.2			177	18.7	52	33.9	5	0.1			10	14.1				
22-23.9	152	17.0	41	0.6			554	58.5	28	18.3	2	0.1			2	2.8				
24-25.9	265	29.7	73	1.0	3	0.1	146	15.4	2	1.3	8	0.2			6	8.5	2	4.1		
26-27.9	203	22.8	164	2.3	6	0.3	2	0.2			11	0.3			9	12.7	4	8.2		
28-29.9	108	12.1	393	5.6	22	1.1	2	0.2	1	0.7	19	0.5	3	3.8	11	15.5	1	2.0	2	6.4
30-31.9	30	3.4	931	13.2	36	1.7	5	0.5			22	0.6	1	1.3	5	7.1	11	22.5	8	25.7
32-33.9	17	2.0	2,682	38.0	157	7.6	8	0.9			138	3.5	2	2.5	8	11.3	15	30.6	3	9.7
34-35.9	9	1.0	1,734	24.5	636	30.8	2	0.2			2,090	52.9	21	26.6	3	4.2	7	14.3	6	19.4
36-37.9	4	0.4	564	8.0	357	17.3					855	21.6	8	10.1	2	2.8	2	4.1	5	16.2
38-39.9	4	0.4	353	5.0	441	21.4					282	7.2	8	10.1	3	4.2	3	6.1	4	16.2
40-41.9			60	0.8	283	13.7					81	2.0	22	27.8	2	2.8	2	4.1		
42-43.9			30	0.4	74	3.6				0.7	32	0.8	2	2.5					2	6.4
44-45.9			17	0.2	42	2.0	1	0.2			34	0.9	3	3.8						
46-47.9			2	0.0	3	0.1	1	0.1			85	2.2	1	1.3	1	1.4				
48-49.9			2	0.0	1	0.1					127	3.2	1	1.3			1	2.0		
50-51.9			1	0.0					1	0.7	44	1.1	1	1.3						
52 and over			1	0.0	2	0.1	4	0.9			96	2.4	6	7.6						
TOTAL	892	100.0	7,073	100.0	2,065	100.0	942	100.0	152	100.0	3,943	100.0	79	100.0	71	100.00	49	100.0	30	100.0

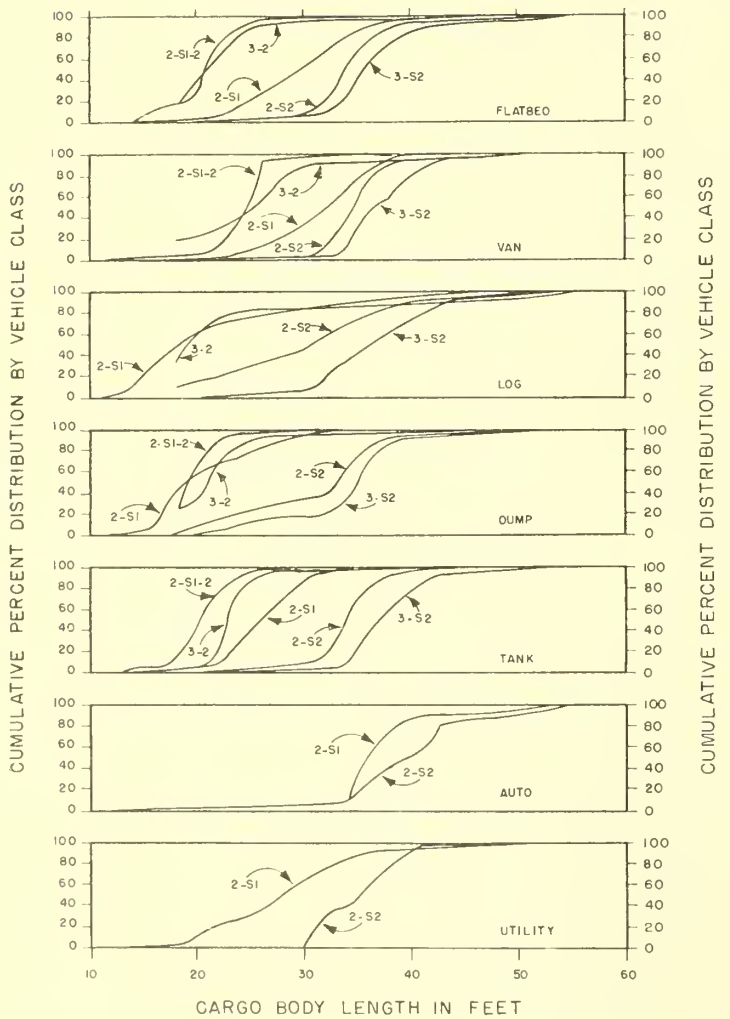

Figure 7.—Cumulative percentage distribution by vehicle type and cargo body length.

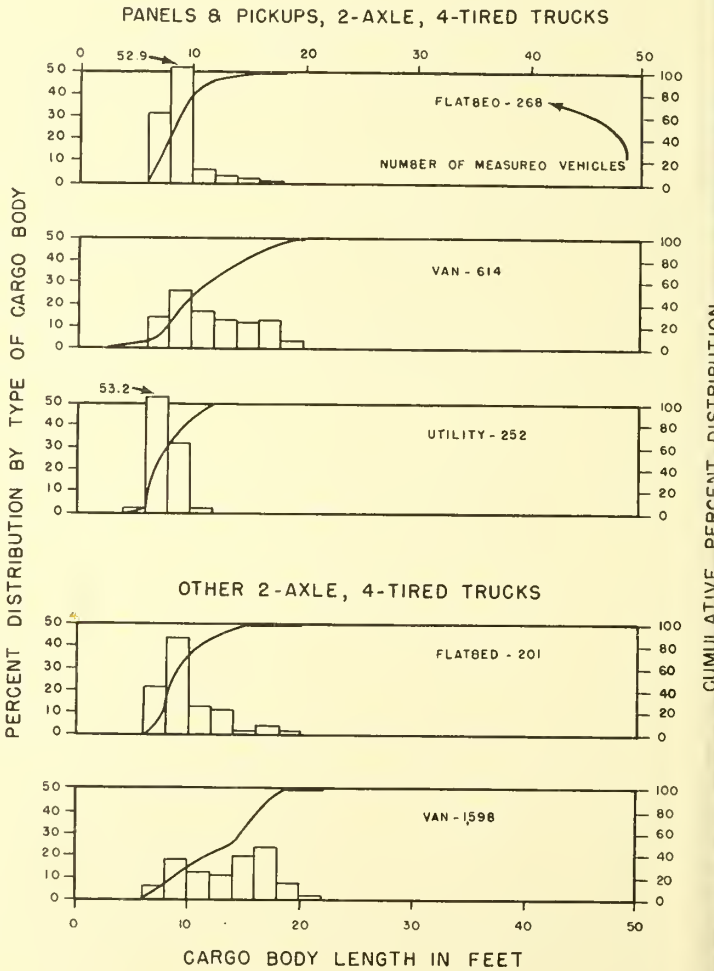

Figure 8.—Distribution of cargo body lengths, 2-axle, 4-tire motortrucks.

Considerable differences were noted in the length distributions of the different types of cargo bodies of 3-axle trucks, figure 10. Lengths of flatbed and van bodies were predominantly in the range of 16 to 22 feet, and lengths of log and tank bodies were mostly in the 14- to 20-foot range. Nearly two-thirds of the dump trucks and 85 percent of the ready-mix concrete trucks were equipped with cargo bodies 12 to 16 feet long, tables 3 and 4.

Empty Vehicle Weights

Trailer combinations

Empty weights were obtained for 27,144 trailer combinations for the five classifications for which the greatest number of trailer combinations occurred—2–S1, 2–S2, 3–S2, 3–2, and 2–S1–2—and are shown in table 5. The weighted average empty weights by class of combination and type of cargo body provide a means of computing average payload weights when average loaded gross weights are known. The empty 2–S2 combinations on the average weighed about 5,000 pounds more than the 2–S1 empty van combinations. Other variations in empty weights between these three classes of combinations and the six types of cargo bodies are shown in table 5. Sometimes the sample of vehicles weighed was small, and averages computed from these data are not as reliable as data might have been if a larger sample could have been obtained. The sizes of the samples are shown in table 5 for use in evaluating the reliability of the data for average empty weights.

In figure 11, average empty weights of trailer combinations have been arranged by cargo body types to show the variations in weight of the same body type for the five main combination classes. Similarly, in figure 12, average empty weights have been arranged by the five main combination classes to show the variations in weight for the different cargo body types.

Single-unit trucks

The four classes of single-unit trucks weighed and measured were panels and pickups having 4 tires, other 2-axle trucks having 4 tires, 2-axle trucks having 6 tires, and 3-axle trucks. Data collected are recorded in table 6. The total number of these types of trucks observed was 23,844. Empty weights averaged 4,800 pounds for pickup trucks and 6,100 pounds for panel trucks. Other 2-axle, 4-tired trucks, having van cargo bodies, on the average had empty weights of only about 300 pounds more than the panel trucks. Two-axle trucks equipped with 6 tires had empty weights that were approximately 3,000 pounds heavier than trucks having 4 tires. Empty weights of 3-axle flatbed, van, and dump trucks ranged between 15,000 and 16,000 pounds; and empty weights of tank trucks averaged about 19,000 pounds. Ready-mixed concrete trucks and utility trucks weighed empty 22,500 and 25,000 pounds, respectively, equipment was a regular part of their empty weight.

In figure 13, average empty weights have been arranged by cargo body types to show the differences in weight of the same body type for the four different vehicle classes. Similarly, in figure 14, average empty weights have been arranged by the four vehicle classes to show the differences in weights for the eight cargo body types.

Table 3.—Cargo body lengths of single-unit trucks, 16 States

Cargo body length	Panels, pickups, 4-tired trucks		2-axle, 4-tired trucks		2-axle, 6-tired trucks		3-axle trucks	
				FLATBED				
Feet	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*
Under 6.0			2	1.0	22	0.2		
6–7.9	87	32.5	43	21.4	68	0.6		
8–9.9	142	52.9	90	44.8	921	8.1	1	0.1
10–11.9	17	6.3	26	12.9	1,023	9.0	11	0.9
12–13.9	9	3.4	24	11.9	3,983	35.1	70	5.8
14–15.9	8	3.0	4	2.0	2,714	24.0	118	9.7
16–17.9	2	0.7	8	4.0	1,511	13.3	342	28.1
18–19.9	1	0.4	4	2.0	589	5.2	309	25.4
20–21.9					254	2.2	233	19.2
22–23.9					122	1.1	60	4.9
24–25.9					62	0.5	29	2.4
26–27.9	1	0.4			39	0.3	12	1.0
28–29.9					15	0.1	12	1.0
30–31.9					8	0.1	4	0.3
32–33.9					6	0.1	8	0.7
34–35.9					13	0.1	4	0.3
36–41.9	1	0.4			4	0.0	2	0.2
Total	268	100.0	201	100.0	11,354	100.0	1,215	100.0
				VAN				
Under 4.0	3	0.5	1	0.1				
4–5.9	1	0.2	3	0.2	39	0.1		
6–7.9	90	14.7	91	5.7	136	0.4		
8–9.9	160	26.0	289	18.0	1,857	6.0	6	0.4
10–11.9	105	17.0	206	12.9	2,288	7.3	10	0.7
12–13.9	77	12.5	197	12.3	9,888	31.7	43	2.8
14–15.9	71	11.6	308	19.3	7,960	25.6	103	6.8
16–17.9	79	12.9	379	23.6	5,251	17.0	319	20.8
18–19.9	27	4.4	105	6.6	1,988	6.4	446	29.2
20–21.9			9	0.6	712	2.3	345	22.6
22–23.9			6	0.4	391	1.3	130	8.5
24–25.9	1	0.2	3	0.2	258	0.8	74	4.9
26–27.9					161	0.5	15	1.0
28–29.9					86	0.3	13	0.9
30–31.9					33	0.1	6	0.4
32–33.9					40	0.1	4	0.3
34–35.9					34	0.1	6	0.4
36–37.9			1	0.1	14	0.0	1	0.1
38–51.9					10	0.0	2	0.2
Total	614	100.0	1,598	100.0	31,146	100.0	1,523	100.0
				LOG				
Under 6.0	NA	NA	NA	NA	3	0.5		
6–7.9	NA	NA	NA	NA	1	0.2		
8–9.9	NA	NA	NA	NA	12	2.1	2	0.5
10–11.9	NA	NA	NA	NA	35	6.1	15	3.6
12–13.9	NA	NA	NA	NA	276	48.2	38	9.1
14–15.9	NA	NA	NA	NA	158	27.6	76	18.2
16–17.9	NA	NA	NA	NA	55	9.6	127	30.3
18–19.9	NA	NA	NA	NA	15	2.6	98	23.4
20–21.9	NA	NA	NA	NA	7	1.2	37	8.9
22–23.9	NA	NA	NA	NA	6	1.0	8	1.9
24–35.9	NA	NA	NA	NA	5	0.9	17	4.1
Total	NA	NA	NA	NA	573	100.0	418	100.0
				DUMP				
Under 6.0	NA	NA	NA	NA	20	0.3		
6–7.9	NA	NA	NA	NA	228	3.2		
8–9.9	NA	NA	NA	NA	2,952	41.7	75	2.8
10–11.9	NA	NA	NA	NA	1,587	22.4	444	16.5
12–13.9	NA	NA	NA	NA	1,338	19.0	1,028	38.3
14–15.9	NA	NA	NA	NA	598	8.4	642	24.0
16–17.9	NA	NA	NA	NA	245	3.5	225	8.4
18–19.9	NA	NA	NA	NA	76	1.1	167	6.2
20–21.9	NA	NA	NA	NA	16	0.2	60	2.2
22–23.9	NA	NA	NA	NA	6	0.1	26	1.0
24–35.9	NA	NA	NA	NA	15	0.1	18	0.6
Total	NA	NA	NA	NA	7,081	100.0	2,685	100.0

Average Payload Weights of Trailer Combinations

The average payload weights shown in table 9 were derived by subtracting the average

Table 4.—Number and percent of length of cargo bodies of single-unit trucks, 46 States

Cargo body length	2-axles, 6-tired		3-axle trucks		2-axles, 6-tired		3-axle trucks		Panels and pickups, 4-tired		2-axles, 6-tired		3-axle trucks	
	TANK				CONCRETE				UTILITY					
Feet	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*	*Number*	*Percent*
Under 6.0	4	0.1							4	1.6	22	1.4		
6–7.9	3	0.1			1	1.1			134	53.2	56	3.7		
8–9.9	45	1.4			6	7.0	3	0.3	81	32.1	335	21.9	13	5.6
10–11.9	166	5.3	3	1.1	25	28.7	54	5.9	28	11.1	269	17.6	23	9.9
12–13.9	1,115	35.4	14	5.4	44	50.6	384	41.6	4	1.6	340	22.2	34	14.7
14–15.9	1,315	41.8	66	25.3	5	5.8	396	42.9			214	14.0	28	12.1
16–17.9	396	12.6	71	27.2	3	3.5	63	6.8	1	0.4	124	8.1	30	12.9
18–19.9	78	2.5	63	24.2	1	1.1	10	1.1			83	5.4	39	16.8
20–21.9	11	0.3	35	13.4	1	1.1	3	0.3			24	1.6	24	10.3
22–23.9	6	0.2	4	1.5			6	0.7			13	0.8	9	3.9
24–25.9	6	0.2	3	1.1	1	1.1	2	0.2			18	1.2	10	4.3
26–27.9	1	0.0					1	0.1			14	0.9	5	2.2
28–29.9											5	0.3	5	2.2
30–31.9			1	0.4							6	0.4	1	0.4
32–33.9	1	0.0									2	0.1	1	0.4
34–35.9	2	0.1					1	0.1			3	0.2	1	0.4
36–37.9	1	0.0									2	0.1		
38–39.9											1	0.1		
40–41.9	1	0.0												
42–43.9 [1]			1	0.4										
44–45.9													1	0.4
46–47.9													2	0.9
48–49.9													1	0.4
50 and over													5	2.2
TOTAL	3,151	100.0	261	100.0	87	100.0	923	100.0	252	100.0	1,531	100.0	232	100.0

[1] For the tank cargo bodies, length is 42 feet and over.

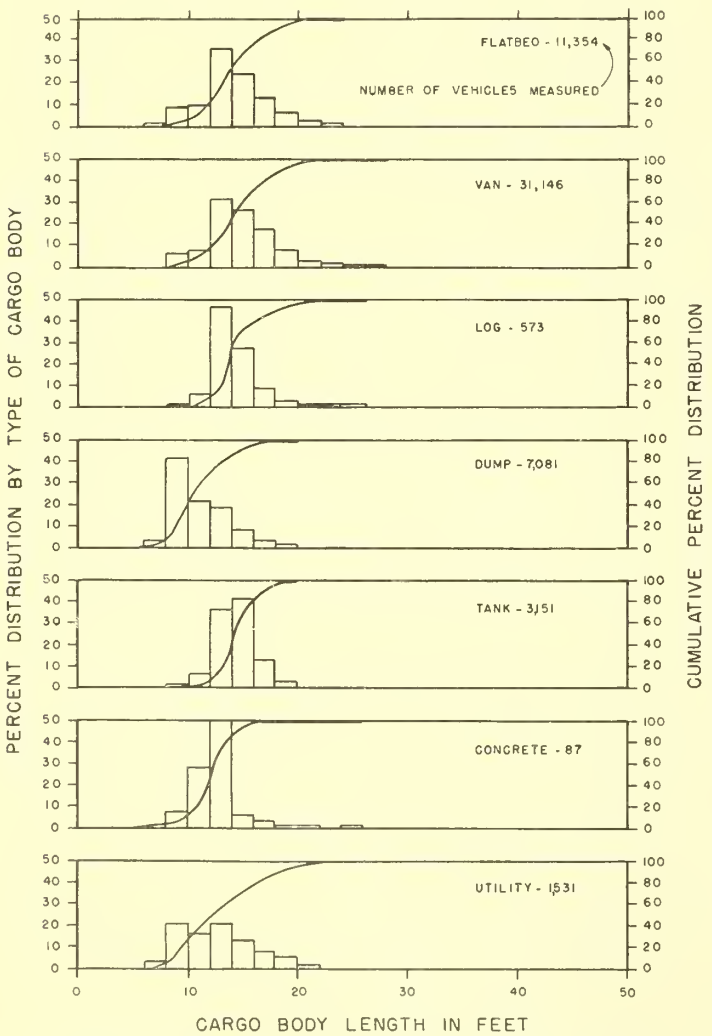

Figure 9.—Distribution of cargo body lengths, 6-tired motortrucks.

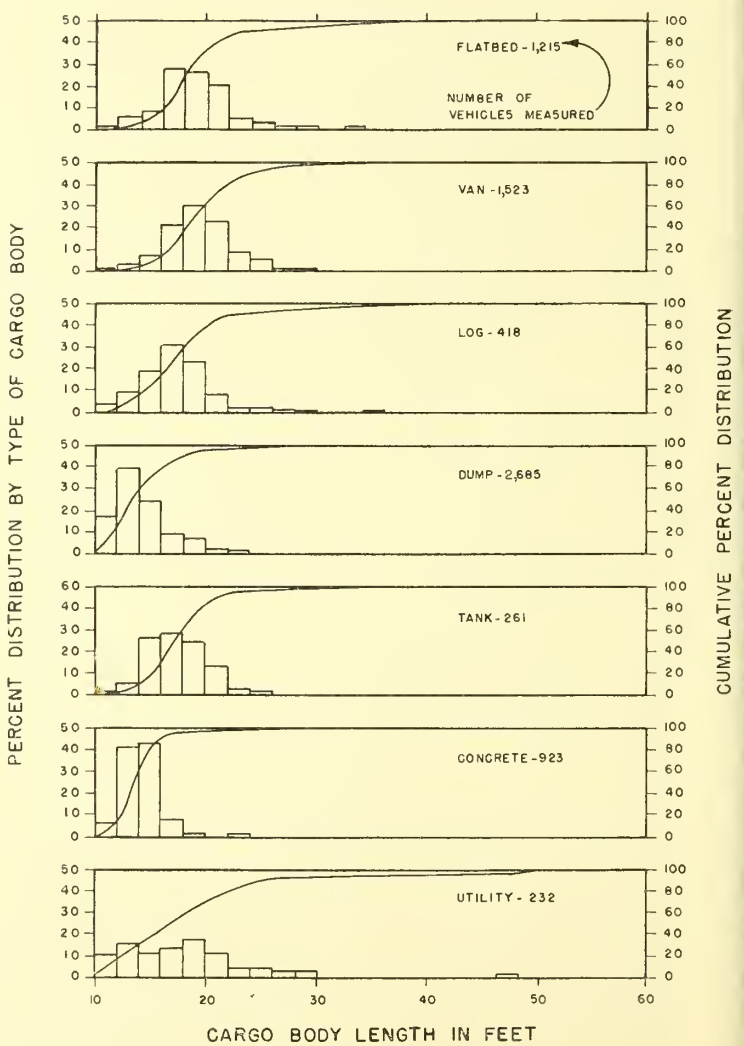

Figure 10.—Distribution of cargo body lengths, 3-axle motortrucks.

Table 5.—Average empty weights of trailer combinations by length and type of cargo body, 46 States, 1959

Trailer body length	Flatbed	Van	Log	Dump	Tank	Auto	Utility
			2–S1				
Feet	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*
10–11.9		23,600	11,500				
12–13.9		17,500	9,400	18,300			
14–15.9	16,000	17,900	9,600	15,500		20,000	
16–17.9	14,200	18,000	10,500	16,800	18,100	12,600	21,400
18–19.9	17,300	18,300	10,700	15,700	19,500		12,200
20–21.9	15,200	17,800	10,800	15,400	19,900		21,200
22–23.9	16,500	18,600	12,300	16,400	20,200		
24–25.9	17,000	18,900	13,400	15,600	19,400	17,700	19,600
26–27.9	16,700	19,100	11,300	15,300	22,100	20,600	17,000
28–29.9	17,500	19,300		16,500	22,200	18,100	22,700
30–31.9	18,300	20,400	12,400	16,700	22,700	20,100	22,100
32–33.9	18,800	20,400	15,000	20,000	21,100	19,900	20,000
34–35.9	18,200	21,200	11,700		17,000	19,000	
36–37.9	17,300	20,900	12,300	16,800		18,600	
38–39.9	17,300	21,300	11,000		19,800	19,300	21,800
40–41.9	16,400	21,500	13,500			20,100	17,800
42–43.9	20,400	23,400				19,300	
44–45.9	16,000	17,900				19,000	
46–47.9		22,300	10,100			20,400	
48–49.9			10,400			21,300	
50–51.9						20,700	
52 and over		17,700				22,100	
Weighted Average	17,500	20,100	10,600	16,200	20,600	19,200	19,900
Vehicles Weighed	866	3,447	216	337	395	1,672	16
			2–S2				
Under 16	25,700	25,700		19,900			NA
16–17.9	18,900	25,100	14,600	21,200			NA
18–19.9		19,000	16,200	23,300	24,200		NA
20–21.9	15,000	27,000	13,000	24,500	24,400		NA
22–23.9	25,400	24,300	16,900	25,700	31,400		NA
24–25.9	24,400	25,500	17,500	26,200	25,200		NA
26–27.9	22,300	23,100	19,700	24,800	25,000		NA
28–29.9	23,900	24,400	19,300	23,800	25,700		NA
30–31.9	22,100	24,100	20,200	24,000	25,400		NA
32–33.9	22,300	24,200	18,700	24,700	24,400		NA
34–35.9	22,500	25,100	20,400	22,500	24,700	21,600	NA
36–37.9	22,300	25,400	20,300	23,400	25,200	20,100	NA
38–39.9	22,900	25,400	18,000	22,400	25,000	22,600	NA
40–41.9	22,300	25,900	19,100	21,400	25,600	24,400	NA
42–43.9	21,700	26,300		21,600	23,600		NA
44–45.9	23,200	25,800		19,800	24,100		NA
46–47.9	25,300	24,000		21,200			NA
Weighted Average	22,500	24,800	17,800	23,900	24,800	21,800	NA
Vehicles Weighed	2,925	8,653	166	885	3,180	25	NA
			3–S2				
Under 18	37,800					NA	
18–19.9	32,800	17,100		26,300		NA	
20–21.9	38,000	23,400	22,100	28,300		NA	
22–23.9	28,400	27,300		27,900		NA	
24–25.9	35,600	30,800		30,500		NA	
26–27.9	31,600	22,300	24,600	29,300	32,200	NA	
28–29.9	32,300	25,400		34,900	29,200	NA	32,900
30–31.9	29,100	30,000	19,800	27,300	33,000	NA	25,500
32–33.9	30,800	28,000		27,400	30,900	NA	25,000
34–35.9	31,000	31,100	24,500	29,300	27,600	NA	25,800
36–37.9	32,500	30,500	22,500	27,500	28,800	NA	
38–39.9	30,300	31,400	24,000	28,200	27,700	NA	
40–41.9	28,500	31,500		27,200	25,000	NA	
42–43.9	31,900	33,300		26,600	26,700	NA	
44–45.9	31,100	30,600		25,600	25,800	NA	
46–47.9	30,500	30,600		31,000		NA	
48–49.9		34,400		23,000		NA	
50–51.9		39,600				NA	
52 and over		33,300		27,000		NA	
Weighted Average	31,000	30,700	23,500	28,500	27,800	NA	26,400
Vehicles Weighed	611	1,522	14	212	973	NA	8
			3–2				
Weighted Average	29,400	28,600	26,300	31,200	28,400	NA	NA
Vehicles Weighed	156	89	6	27	415	NA	NA
			2–S1–2				
Weighted Average	28,000	33,000	NA	28,800	32,700	NA	NA
Vehicles Weighed	80	77	NA	98	73	NA	NA

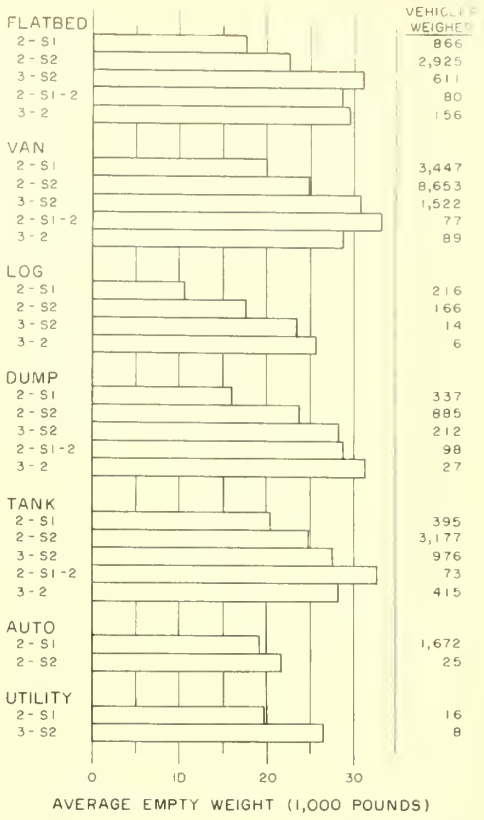

Figure 11.—Average empty weights of trailer combinations by vehicle and cargo body types.

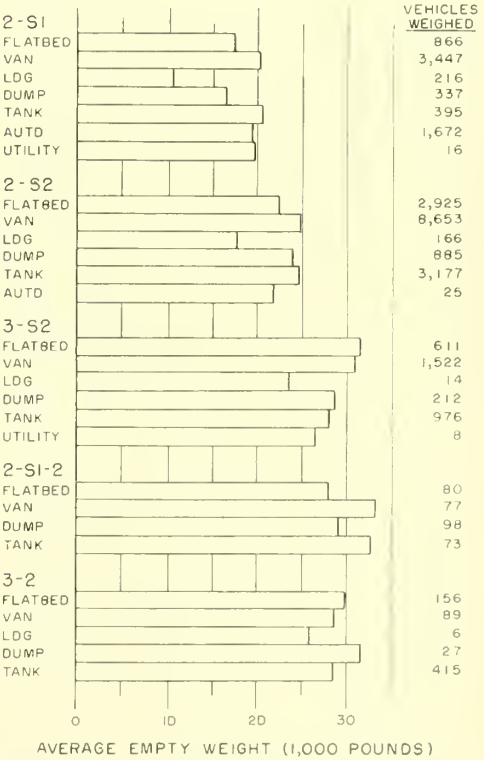

Figure 12.—Average empty weights of trailer combinations by cargo body and vehicle types.

Table 6.—Average empty weights of single-unit trucks by length and type of cargo body, 16 States, 1959

Cargo body length	Flatbed	Van	Log	Dump	Tank	Auto	Concrete	Utility
				PANELS AND PICKUPS, 4-TIRED				
Feet	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*
6.0–7.9	4,200	4,600	NA	4,600	NA	NA	NA	4,100
8.0–9.9	4,800	5,400	NA	5,800	NA	NA	NA	5,100
10.0–11.9	5,400	5,700	NA	5,600	NA	NA	NA	5,100
12.0–13.9	7,000	6,600	NA	8,400	NA	NA	NA	----------
14.0–15.9	7,200	7,900	NA	----------	NA	NA	NA	----------
16.0–17.9	10,000	10,400	NA	----------	NA	NA	NA	----------
18.0–19.9	----------	6,400	NA	----------	NA	NA	NA	----------
Weighted average	4,800	6,100	NA	5,700	NA	NA	NA	4,900
Vehicles weighed	144	218	NA	20	NA	NA	NA	26
				OTHER 2-AXLE, 4-TIRED				
Under 6.0	----------	6,500	NA	NA	NA	NA	NA	----------
6.0–7.9	4,500	4,600	NA	NA	NA	NA	NA	4,600
8.0–9.9	5,000	5,000	NA	NA	NA	NA	NA	6,500
10.0–11.9	5,500	5,800	NA	NA	NA	NA	NA	10,000
12.0–13.9	8,400	7,300	NA	NA	NA	NA	NA	5,200
14.0–15.9	6,200	6,300	NA	NA	NA	NA	NA	----------
16.0–17.9	7,700	6,600	NA	NA	NA	NA	NA	----------
18.0–19.9	7,200	6,800	NA	NA	NA	NA	NA	----------
20.0–21.9	----------	7,600	NA	NA	NA	NA	NA	----------
22.0–23.9	----------	8,600	NA	NA	NA	NA	NA	----------
Weighted average	5,400	6,400	NA	NA	NA	NA	NA	7,500
Vehicles weighed	110	423	NA	NA	NA	NA	NA	11
				2-AXLE, 6-TIRED				
Under 6.0	7,400	7,900	9,800	9,200	8,800	5,900	----------	6,000
6.0–7.9	6,500	6,900	7,200	9,900	10,400	----------	----------	7,500
8.0–9.9	6,200	6,600	7,200	9,900	8,100	----------	11,800	8,600
10.0–11.9	7,100	7,600	7,200	9,900	9,700	----------	14,000	9,600
12.0–13.9	7,700	8,800	7,200	9,100	10,600	8,800	14,200	10,500
14.0–15.9	8,400	9,800	7,500	9,700	11,400	9,600	15,400	12,600
16.0–17.9	9,200	10,300	8,700	9,500	13,400	12,500	13,300	12,200
18.0–19.9	9,900	11,000	10,400	11,500	14,400	----------	----------	13,400
20.0–21.9	10,400	11,800	9,100	9,400	16,100	----------	----------	12,100
22.0–23.9	9,600	11,900	8,000	----------	15,600	----------	----------	5,900
24.0–25.9	11,000	12,700	7,600	----------	19,200	----------	22,000	10,800
26.0–27.9	9,400	12,700	----------	12,600	----------	----------	----------	10,900
28.0–29.9	10,200	13,700	----------	----------	----------	----------	----------	8,400
30.0–31.9	10,800	14,200	----------	----------	----------	----------	----------	10,800
32.0–33.9	11,000	13,200	----------	6,500	----------	----------	----------	----------
34.0–35.9	9,800	16,500	----------	8,700	11,000	----------	----------	17,500
36.0–37.9	8,100	9,300	----------	8,400	----------	----------	----------	----------
38.0–39.9	----------	----------	----------	----------	----------	----------	----------	----------
40.0–41.9	----------	----------	----------	----------	----------	----------	----------	----------
42.0–43.9	----------	7,000	----------	----------	----------	----------	----------	----------
44.0–45.9	----------	21,700	----------	----------	----------	----------	----------	----------
Weighted average	8,000	9,300	7,600	9,700	11,300	9,100	14,300	10,200
Vehicles weighed	4,901	9,479	337	3,799	967	7	33	429
				3-AXLE TRUCKS				
Under 6.0	----------	----------	25,100	16,400	----------	NA	----------	24,700
6.0–7.9	----------	12,600	24,200	17,200	----------	NA	----------	----------
8.0–9.9	----------	16,700	----------	12,400	----------	NA	26,200	25,500
10.0–11.9	14,300	12,800	22,500	13,800	----------	NA	20,900	22,300
12.0–13.9	14,300	15,700	22,500	16,900	13,500	NA	20,900	20,400
14.0–15.9	13,500	15,000	24,000	17,600	16,400	NA	23,400	23,000
16.0–17.9	13,900	13,600	13,700	18,200	18,500	NA	26,600	23,100
18.0–19.9	14,400	15,300	16,900	16,800	19,000	NA	20,500	26,100
20.0–21.9	15,400	15,300	22,200	16,400	27,100	NA	----------	18,100
22.0–23.9	21,400	17,000	20,800	17,300	23,400	NA	20,300	36,400
24.0–25.9	20,400	15,200	12,900	14,200	25,600	NA	----------	40,200
26.0–27.9	30,400	15,700	15,100	17,700	----------	NA	26,700	29,800
28.0–29.9	35,400	15,200	18,800	----------	----------	NA	----------	21,400
30.0–31.9	----------	19,400	----------	13,800	----------	NA	----------	----------
32.0–33.9	18,000	14,700	----------	15,400	----------	NA	----------	----------
34.0–35.9	12,200	17,000	----------	----------	----------	NA	25,800	----------
36.0–37.9	----------	----------	----------	----------	----------	NA	----------	----------
38.0–39.9	----------	28,000	----------	----------	----------	NA	----------	----------
40.0–41.9	----------	----------	----------	----------	----------	NA	----------	----------
42.0–43.9	----------	----------	----------	----------	----------	NA	----------	----------
44.0–45.9	----------	----------	----------	----------	----------	NA	----------	----------
46.0–47.9	----------	----------	----------	----------	----------	NA	----------	52,700
48 and over	----------	----------	----------	----------	----------	NA	----------	63,000
Weighted average	15,100	15,200	19,600	16,600	18,900	NA	22,500	25,000
Vehicles weighed	485	564	137	1,232	96	NA	361	65
TOTAL VEHICLES WEIGHED	5,640	10,684	474	5,051	1,063	NA	394	531

empty weights given in table 5 from average loaded gross weights given in tables 7 and 8. Although there is little correlation between cargo body length and average payload weights, a considerable difference is shown in average payload weights as between different combination classes and different cargo body types. For example, the 2–S1 flatbed combinations had an average payload of 15,000 pounds but 2–S1 van combinations carried average payloads of about 11,000 pounds. Corresponding average payload figures for the 2–S2 combinations were 24,000 and 22,000 pounds, and for the 3–S2 combinations 28,000 and 26,000 pounds.

The 2–S2 combinations for all body types except auto and utility bodies, operated on the average with gross vehicle weights of about 15,000 pounds more than 2–S1 combinations with the same body types. The 3–S2 combination having flatbed, van, and tank body types operated with gross vehicle weights between 10,000 and 12,000 pounds more, on the average, than the 2–S2 combinations having the same body types. The 3–S2 dump combination gross vehicle weights, on the average, were 14,000 pounds heavier than the 2–S2 dump combination.

In those States where the double cargo body combination is permitted, the addition of a 2-axle full-trailer to the 2–S1 combination caused an average increase in gross vehicle weight of 28,000 pounds for the flatbed and van combinations and of 36,000 to 38,000 pounds more for the dump and tank combinations.

Trailer Lengths Related to Loaded Gross Weights

An analysis was made to determine whether any significant difference existed in lengths of trailer cargo bodies for different gross vehicle weights. For this purpose, the gross weights of the different combination classes, broken down by cargo body types, were arrayed in 10,000-pound intervals of gross vehicle weight. Each 10,000-pound interval was further arrayed as to length of cargo body. The results are shown, in figures 15–17, for the three main combination classes—the 2–S1, 2–S2, and 3–S2 tractor semitrailer combinations having van cargo bodies. The configurations in these figures are similar in weight intervals from 20,000- to 70,000-pound gross vehicle weights. No significant increase in lengths of cargo bodies can be detected as gross weights increased. The median of cargo body lengths of 2–S1 combinations for 10,000-pound weight intervals between 20,000 and 60,000 pounds was 32 feet, and the median cargo body length for the 2–S2 combination was 35 feet. Commodity data not collected in this study, would be needed to further analyze choice of trailer body lengths made by industry.

Effect of Gross Weight Limits on Loaded Gross Weights

Maximum gross weight limits prescribed for permitted classes of trailer combinations by the 45 States and the District of Columbia in 1959 are as enumerated: limits in 7 States were 56,000 to 60,000 pounds, in 16 States were 60,000 to 68,000 pounds, in 18 States were 71,000 to 76,000 pounds, and in 5 States were 78,000 pounds and more. The loaded trailer combinations weighed in these 46 States were grouped by their loaded gross weights into four weight categories. The combinations in each weight category were arranged in 10,000-pound class intervals of gross vehicle weight, and the number of loaded combinations observed in each weight category were converted to a percentage of total loaded combinations observed.

Weights of 3–S2 combinations

Depending upon the axle limits allowed, the 3–S2 combination can legally operate at a gross vehicle weight of 72,000 pounds where 32,000-pound tandem axles are specified, and at about 80,000 pounds where 36,000-pound tandem axles are specified. In figure 18, percentages are shown of loaded trailer combinations of the 3–S2 combination that had van cargo bodies. As gross weight limits increased, a higher percentage of the loads were more than 60,000 pounds. For example, the percentages of combinations above this figure and the maximum gross weights permitted by the States were: 35 percent and 60,000-pound maximum gross weight, nearly 41 percent and 68,000-pound maximum gross weight, 50 percent and 76,000-pound maximum gross weight, and nearly 64 percent and 78,000-pound and more maximum gross weight.

These figures would seem to indicate that from the freight standpoint there was a demand for heavier permitted gross weight in the States limiting it to 60,000 pounds and that this demand was held in check by the low weight limits. The greatest percentage of loaded gross weights in the States having maximum limits of 56,000 to 60,000 pounds occurred in the 50,000–60,000-pound weight bracket, and in the other three groups of States a preference was shown for 60,000- to 70,000-pound gross loads. In similar analysis of the data for 3–S2 flat-bed loaded vehicles, shown in figure 19, the findings paralleled those given for the vehicles with van cargo bodies.

Gross Weights of 2–S1–2 and 3–2 Combinations

The 2–S1–2 trailer combination, if operating at single-axle limitations of 18,000 pounds, would have a gross weight of about 80,000

Table 7.—Average loaded weights of trailer combinations by length and type of cargo body, 46 States, 1959

Trailer body length	Flatbed	Van	Log	Dump	Tank	Auto	Utility
			2–s1				
Feet	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*
Under 10.0		36,300		40,400		42,200	
10.0–11.9	43,400	49,200	31,100			24,800	
12.0–13.9	30,000	24,700	34,600	37,100	15,900	32,700	
14.0–15.9	36,700	31,400	33,000	42,500	38,500		18,200
16.0–17.9	31,600	33,100	34,000	44,100	27,800	26,600	20,000
18.0–19.9	30,400	30,200	33,700	40,400	34,700	32,200	30,100
20.0–21.9	31,800	29,000	39,300	38,700	35,700	26,100	34,300
22.0–23.9	31,100	28,900	38,400	38,600	36,100	34,800	15,600
24.0–25.9	32,500	29,400	30,200	38,800	38,900	29,900	28,800
26.0–27.9	33,900	30,100	29,300	36,500	42,300	28,400	24,100
28.0–29.9	32,100	30,700	36,400	37,500	44,700	32,600	35,500
30.0–31.9	33,300	30,500	21,400	31,800	40,900	35,500	26,500
32.0–33.9	34,200	30,600	32,700	32,400	42,300	32,400	30,600
34.0–35.9	31,000	32,000	36,200	34,500	38,900	33,300	23,500
36.0–37.9	30,300	32,600	37,700	35,800	29,300	33,700	23,800
38.0–39.9	30,800	32,900	34,700	29,900	34,300	33,800	26,800
40.0–41.9	30,600	32,300	26,200	21,200		33,200	37,600
42.0–43.9	30,000	35,100	33,500			32,900	
44.0–45.9		38,500	42,000			35,600	
46.0–47.9		36,400	17,800			36,500	42,000
48.0–49.9	27,400		16,400			34,500	
50.0–51.9	29,500					38,600	
52 and over	46,600	26,400	31,900			39,600	
Weighted average	32,500	30,900	33,700	40,300	39,400	33,800	29,200
Vehicles weighed	1,323	8,720	272	457	497	2,271	55
			2–s2				
Under 10.0		19,200			35,700		
10.0–11.9	46,500	37,300					
12.0–13.9	25,900	38,700	32,300	52,200	50,900		
14.0–15.9	18,600	35,400	47,800	48,800	54,500		
16.0–17.9	16,000		49,600	54,500			7,800
18.0–19.9	36,200	51,900	43,800	58,800	56,600		
20.0–21.9	45,000	46,100	41,400	55,500	46,600		
22.0–23.9	48,300	48,700	46,000	60,600	52,200		
24.0–25.9	44,900	46,500	46,800	57,400	51,600		33,800
26.0–27.9	45,500	46,800	50,500	56,700	53,400		27,400
28.0–29.9	47,000	45,700	47,300	55,700	53,200	45,900	
30.0–31.9	47,800	47,100	51,400	52,600	54,400		38,800
32.0–33.9	47,700	47,300	49,000	53,000	55,100	50,700	39,000
34.0–35.9	47,700	44,400	48,600	53,000	55,700	30,200	48,800
36.0–37.9	46,000	47,300	51,200	56,400	56,600	25,500	38,600
38.0–39.9	46,200	47,200	49,700	56,000	59,000	36,300	40,800
40.0–41.9	46,000	48,300	48,800	52,400	55,800	46,800	41,200
42.0–43.9	46,300	48,500	55,100	55,400	56,300	32,600	
44.0–45.9	46,400	49,100	43,200	59,000	58,400	13,600	
46.0–47.9	43,500	47,000	54,700	51,800	58,700		
48.0–49.9	36,500	46,600	46,700	62,400	65,800		34,200
50.0–51.9	37,900	48,700	46,400		67,800	47,400	
52 and over	36,400	57,200	44,800			27,400	
Weighted average	47,200	47,300	48,500	54,600	55,300	36,900	38,700
Vehicles weighed	4,396	25,752	321	1,526	3,896	54	49
			3–s2				
Under 10.0	63,400	62,200			70,900	NA	
10.0–11.9	62,600		36,700			NA	
12.0–13.9		36,900				NA	
14.0–15.9		20,500	60,700			NA	
16.0–17.9		20,500				NA	
18.0–19.9		20,500	83,600	67,600		NA	
20.0–21.9	66,800	51,900	62,100	71,000		NA	
22.0–23.9	64,300	60,600	64,900	69,500		NA	
24.0–25.9	68,000	59,800		65,200	67,100	NA	
26.0–27.9	65,500	62,100	67,100	79,300	66,400	NA	
28.0–29.9	55,200	60,300	70,400	68,900	59,200	NA	41,700
30.0–31.9	52,600	55,900	71,500	71,500	60,600	NA	43,100
32.0–33.9	55,700	53,300	70,100	65,700	64,800	NA	58,300
34.0–35.9	58,100	57,800	68,900	64,900	66,900	NA	57,800
36.0–37.9	59,200	58,100	70,600	66,800	66,900	NA	65,300
38.0–39.9	60,000	57,000	69,600	65,800	66,500	NA	81,200
40.0–41.9	62,700	57,100	70,200	65,800	65,700	NA	
42.0–43.9	61,900	59,000	70,700	66,800	66,500	NA	84,100
44.0–45.9	63,100	57,100	70,300	61,700	66,000	NA	
46.0–47.9	74,600	50,700	67,900	70,600	58,300	NA	
48.0–49.9	62,200	56,800	64,300	64,800	61,900	NA	
50.0–51.9	68,600	56,000	62,000			NA	
52 and over	62,600	59,200	60,200			NA	
Weighted average	58,800	57,100	70,000	68,500	66,300	NA	61,400
Vehicles weighed	1,041	8,071	657	481	1,089	NA	22

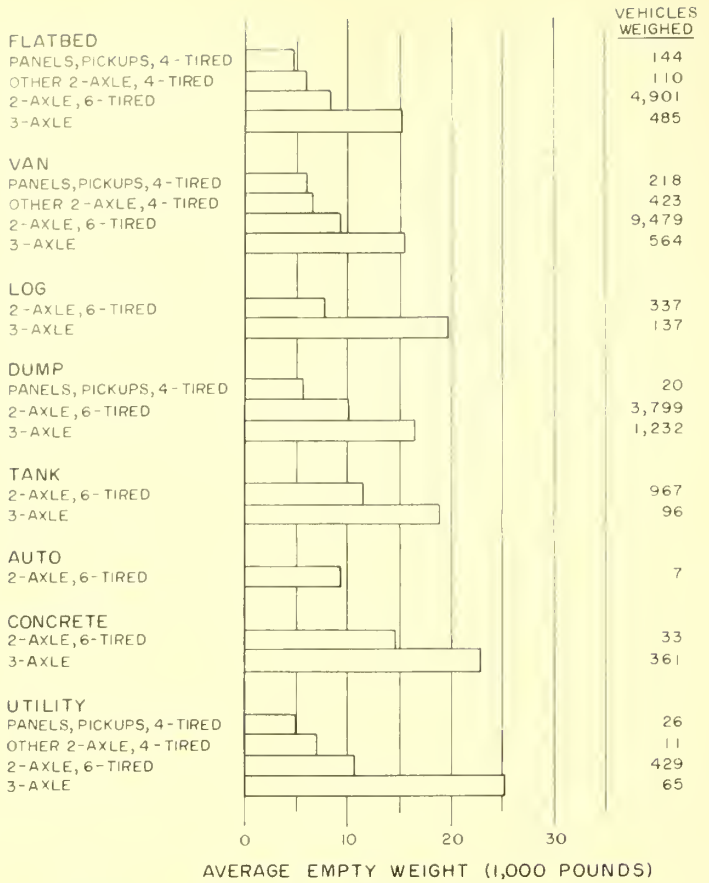

Figure 13.—Average empty weights of single-unit trucks by vehicle and cargo body types.

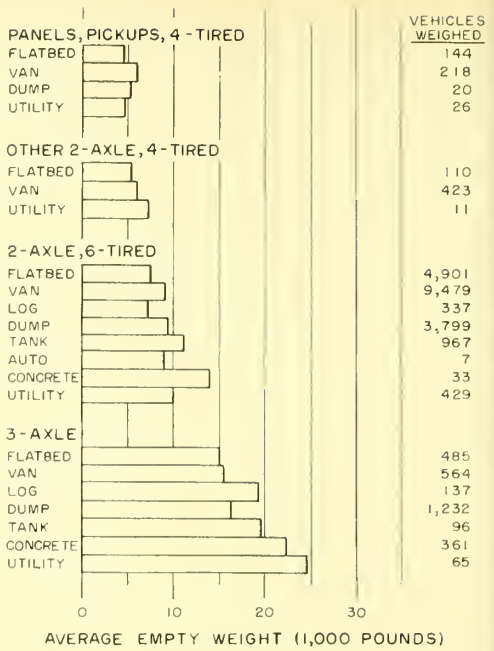

Figure 14.—Average empty weights of single-unit trucks by cargo body and vehicle types.

pounds; and if operating at single-axle limitations of 22,400 pounds, would have a gross weight of about 98,000 pounds. The 3–2 trailer combinations, if operating with 18,000-pound single axles and 32,000-pound tandem axles, would have a maximum gross weight of about 77,000 pounds. The 3–2 combination, if operating with 22,400-pound single axles and 36,000-pound tandem axles, would have a maximum gross weight of about 91,000 pounds. The 2–S1–2 tractor, semitrailer, full-trailer combinations and the 3–2 tractive truck full-trailers combinations were observed mostly in two groups of States—18 States that have maximum weight limits of 71,000 to 76,000 pounds and 5 States that have maximum weight limits of 78,000 pounds and more.

As shown in figure 20, the percentage of 2–S1–2 trailer combinations having gross weights of 80,000 pounds or more was higher in the 5 States having weight limits of 78,000 pounds and more in the 18 States having maximum weight limits of 71,000 to 76,000 pounds. The same trend existed in percentage relationship for the three major body types—flatbed, van, and tank. Similar trends in the relationship of gross weights and the permitted weights were noted for the 3–2 tractive-truck full-trailer combination, figure 21. The percentages for gross weights of combinations of more than 80,000 pounds are shown in table 10.

The data included in figures 20 and 21 and in table 10 indicated that tank cargo body combinations are the ones that can most consistently use the maximum permitted, or higher, gross weights. The two other cargo body types of combinations regularly carried loads that weighed much below the maximum permitted weights. Hence, it may be concluded that not all freight carriers could use to advantage any increase in permitted gross weights. This situation presents a difficult problem in allocating any increased highway construction and maintenance costs for higher load-capacity roadways only to those vehicles that could and would use such increased load-carrying capacities built into a road system. Therefore, the increased roadway costs, occasioned by permitting heavier axle and larger gross weight limits, might not be justified because of possible insufficient use by vehicles carrying heavier loads.

Widths and Heights

During the 1959 truck weight study, the widths of cargo vehicles less than 7 feet wide and heights of cargo vehicles less than 10 feet high were not recorded in most States. Measurement figures were recorded for cargo vehicles of these dimensions and larger. In 1959, Connecticut and Rhode Island permitted widths of 8.5 feet but all other continental States limited widths to 8 feet, exclusive of safety equipment. Out of all the continental States studied, 2 had no height limitations, 2 specified 14.0 feet, 26 specified 13.5 feet, 2 specified 13.0 feet, and 17 specified 12.5 feet. Thus, 30 States in 1959 permitted heights of 13.5 feet or more. As of December 31, 1961, 44 of the continental States had height limitations of 13.5 feet or more and 5 retained the limitation of 12.5 feet.

Because of the trend toward the 8-foot width and 13.5-foot height, measurements taken at truck weight stations have been tabulated to show measurements in excess of these two modal figures. Some of the figures showing measurements greater than the permitted widths and heights probably may be ascribed to special permit loads and to the mounting of tires larger than the 10.00×20 size. When 11.00×20 and larger size tires are placed on highway freight vehicles having body widths of exactly 8 feet, frequently as much as 2 inches of tire may project beyond the body frame on each side. With this in mind, width measurements were separated into intervals of 8.0 to 8.3 feet (8 feet 3.6 inches), 8.4 to 8.5 feet, 8.6 to 9.0 feet, and 9.1 feet and over. Although approximately 10 percent of the total of the trucks having six or more tires and the trailer combinations exceeded the 8-foot width limitation, only about 1 percent of the total of these vehicles exceeded the width of 8 feet 3.6 inches (8.3 feet), as shown in table 11. Approximately 0.3 percent of all trailer combinations and trucks having six or more tires were more than 13.5 feet high. More of the 3–S2 combinations exceeded this height than any other type of vehicle, as shown in table 12.

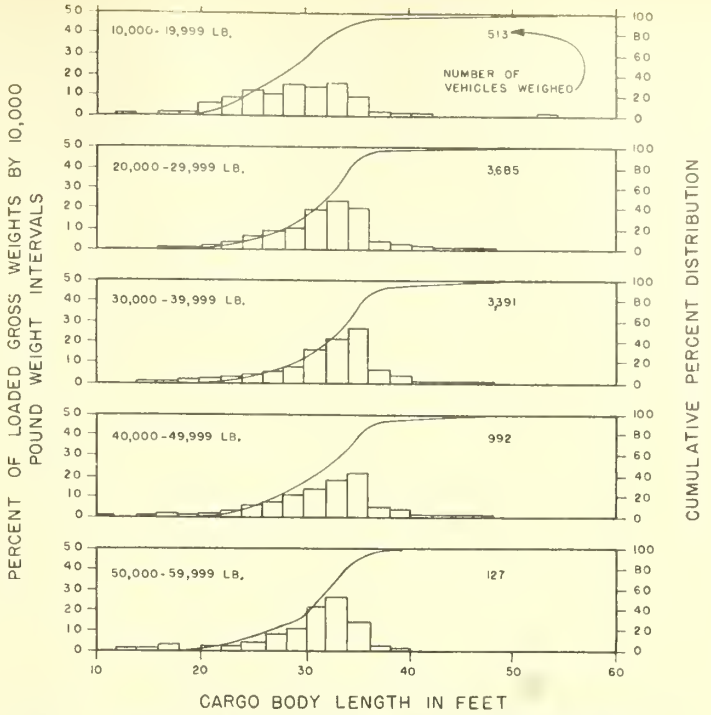

Figure 15.—Distribution of loaded gross weights by lengths of van cargo bodies for 3-axle, tractor semitrailers in the 10,000-pound weight group.

Table 8.—Average loaded weights of trailer combinations by length and type of cargo body, 16 States, 1959

Trailer body length	Flatbed	Van	Log	Dump	Tank
3-2					
Feet	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*
Under 10.0		20,500		74,700	75,700
10.0–11.9			69,400		69,900
12.0–13.9	50,800		78,400	57,600	62,300
14.0–15.9	63,700	30,500	66,900		70,600
16.0–17.9	64,600	48,400	74,400	68,300	67,200
18.0–19.9	68,700	62,000	71,900	75,200	66,000
20.0–21.9	68,900	57,000	73,400	77,200	72,200
22.0–23.9	62,100	51,600	65,200	74,800	71,200
24.0–25.9	57,600	61,300	74,600	88,700	72,800
26.0–27.9	63,000	67,500			72,600
28.0–29.9	71,800	68,700			72,300
30.0–31.9	46,100	16,400			76,000
32.0–33.9		50,100	73,800		78,500
34.0–35.9		47,000			76,000
36.0–37.9		74,500			
38.0–39.9		59,500		60,400	
40.0–41.9		20,500			
42.0–43.9					
44.0–45.9				68,800	73,400
46.0–47.9			67,800		55,700
48.0–49.9	63,800		64,500		
50.0–51.9		45,500			
52 and over	61,800	23,800	70,000		66,600
Weighted average	66,100	56,700	71,800	75,200	71,500
Vehicles weighed	322	111	31	49	527
2-S1-2					
10.0–11.9	78,000		NA		
12.0–13.9	66,800	72,200	NA	74,000	58,600
14.0–15.9	66,300	61,700	NA	77,000	51,100
16.0–17.9	72,600	48,500	NA	78,000	70,200
18.0–19.9	65,200	62,200	NA	78,600	83,000
20.0–21.9	56,200	59,000	NA	75,700	70,800
22.0–23.9	67,600	57,100	NA	88,700	80,200
24.0–25.9	64,500	59,400	NA		80,600
26.0–27.9	62,600	69,400	NA		
28.0–29.9		47,700	NA		40,400
30.0–31.9			NA		
32.0–33.9			NA		
34.0–35.9		46,500	NA		
36.0–37.9			NA	71,400	
38.0–39.9		59,100	NA		
50.0–51.9			NA		55,600
52 and over	84,300	69,300	NA		
Weighted average	61,600	58,600	NA	78,300	74,600
Vehicles weighed	185	317	NA	125	79

Table 9.—Average payload weights of trailer combinations by length and type of cargo body, 46 States, 1959

Trailer body length	Flatbed	Van	Log	Dump	Tank	Auto	Utility
			2-S1				
Feet	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*	*Pounds*
10.0–11.9			19,600				
12.0–13.9			25,200	18,800			
14.0–15.9	20,700	13,500	23,400	27,000			
16.0–17.9	17,400	15,100	23,500	27,300	9,700	14,000	
18.0–19.9	13,100	11,900	23,000	24,700	15,200		17,900
20.0–21.9	16,600	14,000	28,500	23,300	15,800		13,100
22.0–23.9	14,500	12,500	26,100	22,200	15,900		
24.0–25.9	15,500	10,500	16,800	23,200	19,500	12,200	9,200
26.0–27.9	17,200	11,000	18,000	21,200	20,200	7,800	7,100
28.0–29.9	14,600	11,400		21,000	22,500	14,500	12,800
30.0–31.9	15,000	10,100	9,000	15,100	18,200	15,400	4,400
32.0–33.9	15,400	10,200	17,700	12,400	21,200	12,500	10,600
34.0–35.9	12,800	10,800	24,500		21,900	14,300	
36.0–37.9	13,000	11,700	25,400	19,000		15,100	
38.0–39.9	13,500	11,600	23,700		14,500	14,500	5,000
40.0–41.9	14,200	10,800	12,700			13,100	19,800
42.0–43.9	9,600	11,700				13,600	
44.0–45.9		20,600				16,600	
46.0–47.9		14,100	7,700			16,100	
48.0–49.9			6,600			13,200	
50.0–51.9						17,900	
52 and over						17,500	
Weighted average	15,000	10,800	23,100	24,100	18,800	14,600	9,300
Vehicles weighed	1,306	8,705	252	450	490	2,257	40
			2-S2				
16.0–17.9			35,000	33,300			NA
18.0–19.9		32,900	27,600	35,500			NA
20.0–21.9	30,000	19,100	28,400	32,200			NA
22.0–23.9	22,900	24,400	29,100	34,900			NA
24.0–25.9	20,500	21,000	29,300	31,200			NA
26.0–27.9	23,200	23,700	30,800	31,900			NA
28.0–29.9	23,100	21,300	28,000	31,900	27,500		NA
30.0–31.9	25,700	23,000	31,200	28,600			NA
32.0–33.9	25,400	23,100	30,300	28,300	36,700		NA
34.0–35.9	25,200	22,300	28,200	30,500	31,000	8,600	NA
36.0–37.9	23,700	21,900	30,900	33,000		5,400	NA
38.0–39.9	23,300	21,800	31,700	33,600	34,000	13,900	NA
40.0–41.9	23,700	22,400	29,700	31,000	30,200	22,400	NA
42.0–43.9	24,600	27,200		33,800	32,700		NA
44.0–45.9	23,200	23,300		39,200			NA
46.0–47.9	18,200	23,000		30,600			NA
Weighted average	24,700	22,500	30,700	30,700	30,800	15,100	NA
Vehicles weighed	4,363	25,712	271	1,519	2,918	39	NA
			3-S2				
18.0–19.9				41,300		NA	
20.0–21.9	28,800	28,500	40,000	42,700		NA	
22.0–23.9	35,900	33,300		41,600		NA	
24.0–25.9	32,400	29,000		34,700		NA	
26.0–27.9	33,900	39,800	42,500	40,000	34,200	NA	
28.0–29.9	22,900	34,900		34,000	30,000	NA	8,800
30.0–31.9	23,500	25,900	51,700	44,200	27,600	NA	17,600
32.0–33.9	24,900	25,300		38,300	33,900	NA	25,300
34.0–35.9	27,100	26,800	44,400	35,600	39,300	NA	32,000
36.0–37.9	26,700	27,600	48,100	39,300	31,800	NA	
38.0–39.9	29,700	25,600	45,600	37,600	38,800	NA	
40.0–41.9	34,200	25,600		38,600	40,700	NA	
42.0–43.9	30,000	25,700		40,200	39,800	NA	
44.0–45.9	32,000	26,500		36,100	40,200	NA	
46.0–47.9	44,100	20,200		39,600		NA	
48.0–49.9		22,400		41,800		NA	
50.0–51.9		16,400				NA	
52 and over		25,900				NA	
Weighted average	27,800	26,400	46,500	40,000	38,500	NA	22,700
Vehicles weighed	1,022	8,059	384	481	1,084	NA	11
			3-2				
Weighed average	36,700	28,100	45,500	44,000	43,100	NA	NA
Vehicles weighed	315	91	14	43	515	NA	NA
			2-S1-2				
Weighted average	33,600	25,600		49,500	41,800	NA	NA
Vehicles weighed	163	300		122	74	NA	NA

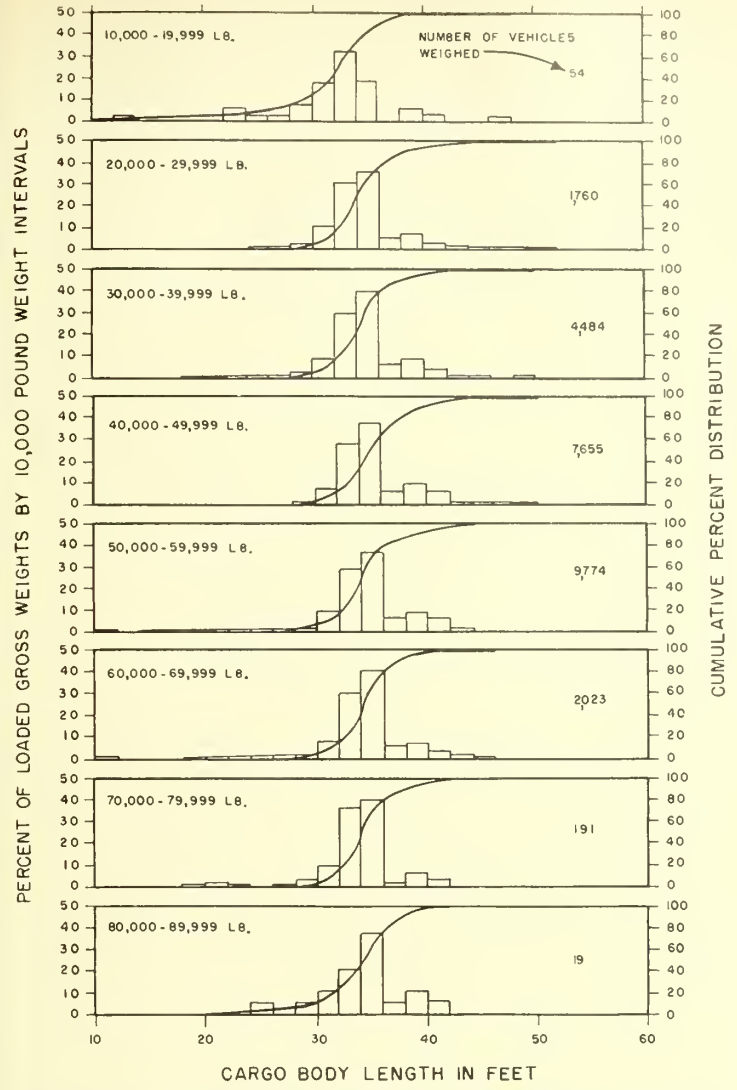

Figure 16.—Distribution of loaded gross weights by lengths of van cargo bodies for 4-axle, tractor semitrailers in the 10,000-pound weight group.

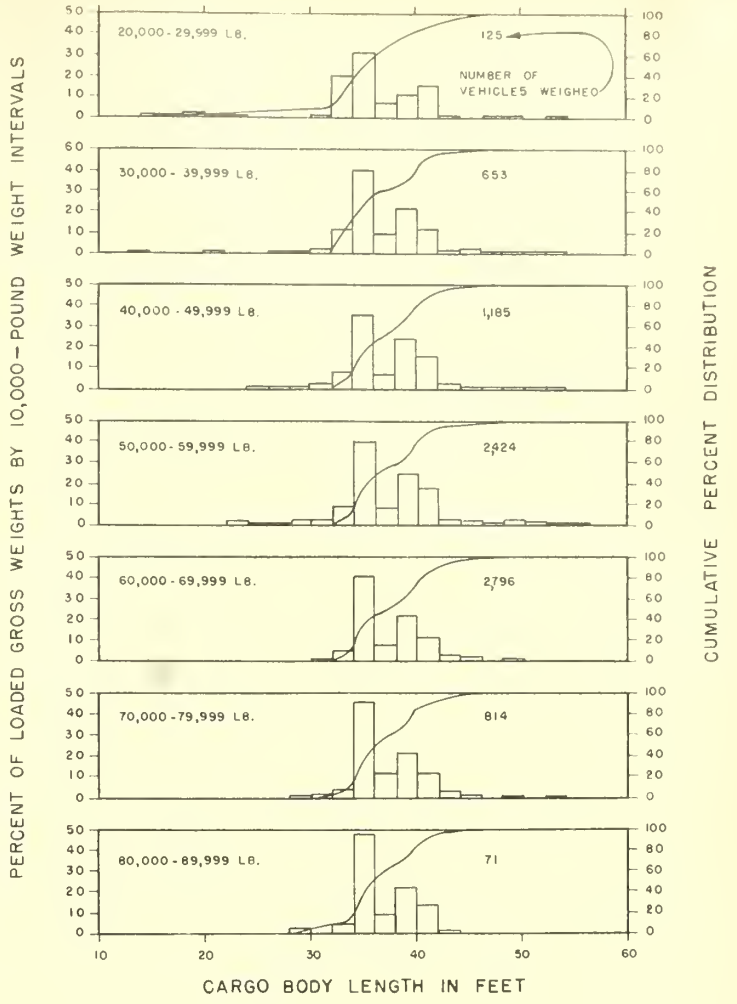

Figure 17.—Distribution of loaded gross weights by lengths of van cargo bodies for 5-axle, tractor semitrailers in the 10,000-pound weight group.

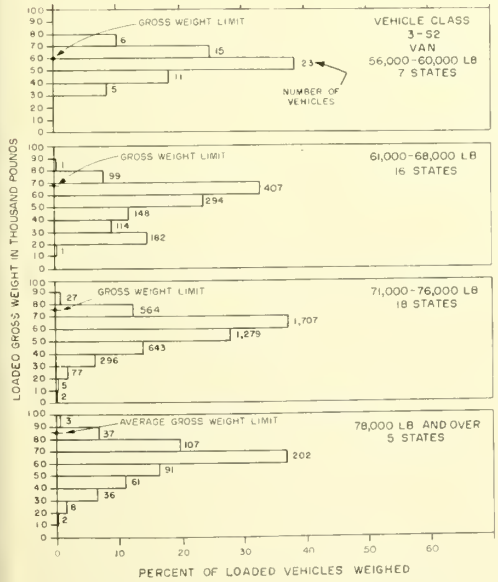
GROSS WEIGHT LIMIT
VEHICLE CLASS 3-S2 VAN 56,000-60,000 LB 7 STATES
NUMBER OF VEHICLES
61,000-68,000 LB 16 STATES
71,000-76,000 LB 18 STATES
AVERAGE GROSS WEIGHT LIMIT
78,000 LB AND OVER 5 STATES
LOADED GROSS WEIGHT IN THOUSAND POUNDS
PERCENT OF LOADED VEHICLES WEIGHED

Figure 18.—Distribution of gross weights by body type and axle classification in relation to permitted weight limits.

Table 10.—Percentages of 2-S1-2 and 3-2 trailer combinations having gross weights of more than 80,000 pounds

Cargo bodies	18 States [1]	5 States [2]
	Percent	*Percent*
2-S1-2 trailer combinations:		
Flatbed	6	25
Van	1	16
Tank	7	45
3-2 trailer combinations:		
Flatbed	0	23
Van	0	5
Tank	1	12

[1] Percentages given are for States where maximum legal weight was 71,000 to 76,000 pounds.
[2] Percentages given are for States where maximum legal weight was 78,000 pounds and more.

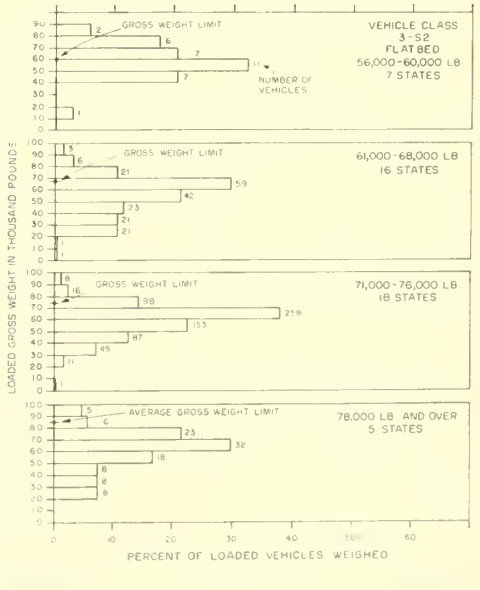
GROSS WEIGHT LIMIT
VEHICLE CLASS 3-S2 FLATBED 56,000-60,000 LB 7 STATES
NUMBER OF VEHICLES
61,000-68,000 LB 16 STATES
71,000-76,000 LB 18 STATES
AVERAGE GROSS WEIGHT LIMIT
78,000 LB AND OVER 5 STATES
LOADED GROSS WEIGHT IN THOUSAND POUNDS
PERCENT OF LOADED VEHICLES WEIGHED

Figure 19.—Distribution of gross weights by body type and axle classification in relation to permitted weight limits.

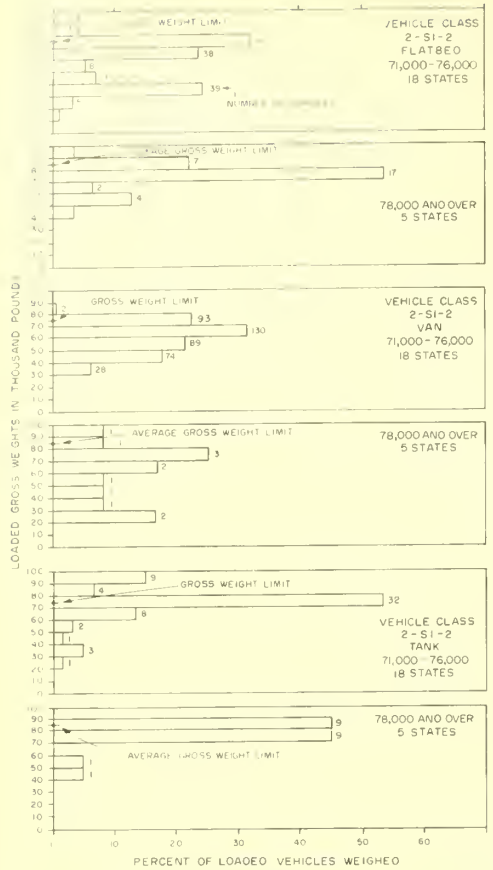

Figure 20.—Distribution of gross weights by body type and axle classification in relation to permitted weight limits.

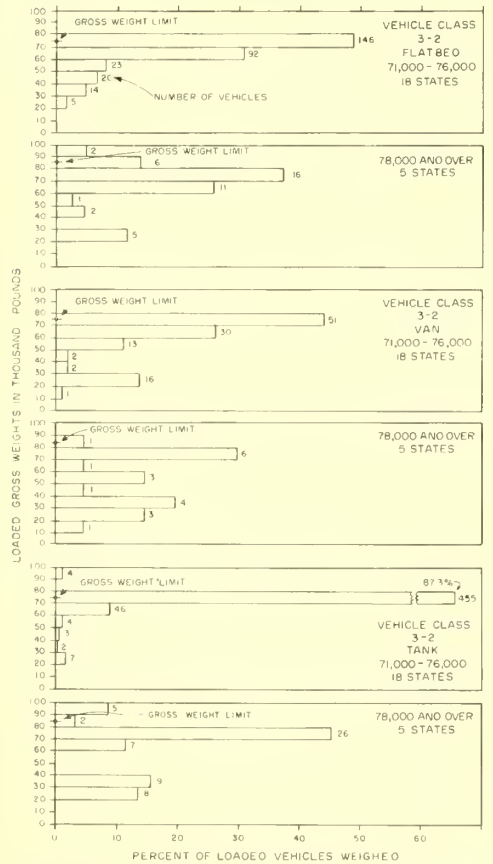

Figure 21.—Distribution of gross weights by body type and axle classification in relation to permitted weight limits.

Table 11.—Number and percent of single-unit trucks and trailer combinations that exceeded 8 feet in width, 46 States, 1959

Cargo or trailer body widths, feet	Flat-bed	Van	Log	Dump	Tank	Auto	Con-crete	Utility	Total
2-axle, 6-tired trucks:									
8.1-8.3	712	1,905	10	255	87	NA	8	66	3,043
8.4-8.5	61	84	3	9	10	NA	3	7	177
8.6-9.0	57	41	1	13	6	NA	-------	4	122
9.1 and over	32	8	-------	5	3	NA	-------	1	49
Over 8.0	862	2,038	14	282	106	NA	11	78	3,391
Over 8.3	150	133	4	27	19	NA	3	12	348
Total measured	11,354	31,146	573	7,081	3,151	NA	87	1,531	54,923
Percent over 8.0	7.6	6.5	2.4	4.0	3.3	NA	12.6	5.0	6.1
Percent over 8.3	1.3	0.4	0.7	0.4	0.6	NA	3.4	0.8	0.6
3-axle, single-unit trucks:									
8.1-8.3	146	125	55	339	11	NA	143	24	843
8.4-8.5	14	5	7	17	3	NA	24	15	85
8.6-9.0	17	1	1	19	15	NA	18	8	79
9.1 and over	7	1	-------	1	-------	NA	1	2	12
Over 8.0	184	132	63	376	29	NA	186	49	1,019
Over 8.3	38	7	8	37	18	NA	43	25	176
Total measured	1,215	1,523	418	2,685	261	NA	923	232	7,257
Percent over 8.0	19.2	8.7	15.1	14.0	11.2	NA	20.2	21.1	14.6
Percent over 8.3	4.0	0.5	1.9	1.4	6.9	NA	4.7	10.8	2.5
2-S1:									
8.1-8.3	229	1,148	42	65	104	360	NA	6	1,954
8.4-8.5	18	43	6	-------	9	11	NA	1	88
8.6-9.0	23	20	2	2	1	7	NA	-------	55
9.1 and over	17	5	3	-------	-------	-------	NA	-------	25
Over 8.0	287	1,216	53	67	114	378	NA	7	2,122
Over 8.3	58	68	11	2	10	18	NA	1	168
Total measured	2,189	12,167	488	794	892	3,943	NA	71	20,544
Percent over 8.0	13.1	10.1	10.9	8.4	12.8	9.6	NA	9.9	10.3
Percent over 8.3	2.6	0.6	2.3	0.3	1.1	0.5	NA	1.4	0.8
2-S2:									
8.1-8.3	807	3,161	74	225	895	7	NA	5	5,174
8.4-8.5	67	85	16	6	64	2	NA	1	241
8.6-9.0	37	51	5	4	28	-------	NA	1	126
9.1 and over	27	8	3	1	2	1	NA	4	46
Over 8.0	938	3,305	98	236	989	10	NA	11	5,587
Over 8.3	131	144	24	11	94	3	NA	6	413
Total measured	7,321	34,405	487	2,411	7,073	79	NA	49	51,825
Percent over 8.0	12.8	9.6	20.2	9.8	14.0	12.7	NA	22.4	10.8
Percent over 8.3	1.8	0.4	5.0	0.5	1.3	3.8	NA	12.2	0.8
3-S2:									
8.1-8.3	278	890	90	68	97	NA	NA	3	1,426
8.4-8.5	15	10	15	7	6	NA	NA	-------	53
8.6-9.0	22	8	21	-------	2	NA	NA	1	54
9.1 and over	27	3	6	2	1	NA	NA	5	44
Over 8.0	342	911	132	77	106	NA	NA	9	1,577
Over 8.3	64	21	42	9	9	NA	NA	6	151
Total measured	1,652	9,593	671	693	2,065	NA	NA	30	14,704
Percent over 8.0	20.7	9.5	20.1	11.1	5.1	NA	NA	40.9	10.7
Percent over 8.3	3.9	0.2	6.4	1.3	0.4	NA	NA	27.3	1.0
2-S1-2:									
8.1-8.3	36	70	NA	32	46	NA	NA	NA	184
8.4-8.5	5	1	NA	1	-------	NA	NA	NA	7
8.6-9.0	4	-------	NA	-------	3	NA	NA	NA	7
9.1 and over	-------	-------	NA	-------	-------	NA	NA	NA	-------
Over 8.0	45	71	NA	33	49	NA	NA	NA	198
Over 8.3	9	1	NA	1	3	NA	NA	NA	14
Total measured	265	394	NA	223	152	NA	NA	NA	1,034
Percent over 8.0	17.0	18.0	NA	14.8	32.2	NA	NA	NA	19.1
Percent over 8.3	3.4	0.3	NA	0.4	2.0	NA	NA	NA	1.4
3-2:									
8.1-8.3	105	30	2	33	164	NA	NA	NA	334
8.4-8.5	1	-------	1	-------	4	NA	NA	NA	6
8.6-9.0	1	-------	2	-------	-------	NA	NA	NA	3
9.1 and over	1	-------	-------	1	-------	NA	NA	NA	2
Over 8.0	108	30	5	34	168	NA	NA	NA	345
Over 8.3	3	-------	3	1	4	NA	NA	NA	11
Total measured	478	200	37	76	942	NA	NA	NA	1,733
Percent over 8.0	22.6	15.0	13.5	44.7	17.8	NA	NA	NA	19.3
Percent over 8.3	0.6	-------	8.1	1.3	0.4	NA	NA	NA	0.6
Other trucks:									
Panels and pickups, 4-tired	268	614	NA	NA	NA	NA	NA	252	1,134
2-axle, 4-tired	201	1,598	NA	NA	NA	NA	NA	-------	1,799
Other combinations:									
2-1	75	NA	NA	NA	NA	NA	NA	NA	75
2-2	78	NA	NA	NA	NA	NA	NA	NA	78
2-S2-2	NA	53	NA	NA	NA	NA	NA	NA	53
3-S1-1	NA	54	NA	NA	NA	NA	NA	NA	54
3-S3-2	NA	55	NA	NA	NA	NA	NA	NA	55
TOTAL MEASURED	25,096	91,802	2,674	13,963	14,536	4,022	1,010	2,165	155,268
7 vehicle classes	24,474	89,428	2,674	13,963	14,536	4,022	1,010	1,913	152,020
Over 8.0	2,766	7,703	365	1,105	1,561	388	197	154	14,239
Over 8.3	453	374	92	88	157	21	46	50	1,281
Percent over 8.0	11.3	8.6	13.6	7.9	10.7	9.6	19.5	8.1	9.4
Percent over 8.3	1.9	0.4	3.4	0.6	1.1	0.5	4.6	2.6	0.8

Table 12.—Number and percent of single-unit trucks and trailer combinations that exceeded 13.5 feet in height, 46 States, 1959

Cargo or trailer body heights, feet	Flat-bed	Van	Log	Dump	Tank	Auto	Con-crete	Utility	Total
2-axle, 6-tired trucks:									
13.6 to 14.0	17			2		NA			19
14.1 to 14.5	7	1				NA			8
14.6 and over	5	1				NA			6
13.6 and over	29	2		2		NA			33
Total measured	11,354	31,146	573	7,081	3,151	NA	87	1,531	54,923
Percent 13.6 and over	0.3					NA			0.1
3-axle trucks:									
13.6 to 14.0	1	2	2			NA		3	8
14.1 to 14.5						NA			
14.6 and over				1		NA			1
13.6 and over	1	2	2	1		NA		3	9
Total measured	1,215	1,523	418	2,685	261	NA	923	232	7,257
Percent 13.6 and over	0.1	0.1	0.5			NA		1.3	0.1
2-S1:									
13.6 to 14.0	14	6				34	NA	1	55
14.1 to 14.5	1					10	NA		11
14.6 and over	3	2				1	NA		6
13.6 and over	18	8				45	NA	1	72
Total measured	2,189	12,167	488	794	892	3,943	NA	71	20,544
Percent 13.6 and over	0.8	0.1				1.1	NA	1.4	0.4
2-S2:									
13.6 to 14.0	38	11	3	1		5	NA		58
14.1 to 14.5	12					2	NA		14
14.6 and over	8	14					NA		22
13.6 and over	58	25	3	1		7	NA		94
Total measured	7,321	34,405	487	2,411	7,073	79	NA	49	51,825
Percent 13.6 aud over	0.8	0.1	0.6			8.9	NA		0.2
3-S2:									
13.6 to 14.0	34	105	11	3	1	NA	NA	1	155
14.1 to 14.5	8	3	3			NA	NA		14
14.6 and over	21	5	3	1		NA	NA	1	31
13.6 and over	63	113	17	4	1	NA	NA	2	200
Total measured	1,652	9,593	671	693	2,065	NA	NA	30	14,704
Percent 13.6 and over	3.8	1.2	2.6	0.6		NA	NA	9.1	1.4
2-S1-2:									
13.6 to 14.0	3	26	NA			NA	NA	NA	29
14.1 to 14.5	1	2	NA			NA	NA	NA	3
14.6 and over			NA			NA	NA	NA	
13.6 and over	4	28	NA			NA	NA	NA	32
Total measured	265	394	NA	223	152	NA	NA	NA	1,034
Percent 13.6 and over	1.5	7.1	NA			NA	NA	NA	3.1
3-2:									
13.6 to 14.0		1				NA	NA	NA	1
14.1 to 14.5		2				NA	NA	NA	2
14.6 and over						NA	NA	NA	
13.6 and over		3				NA	NA	NA	3
Total measured	478	200	37	76	942	NA	NA	NA	1,733
Percent 13.6 and over		1.5				NA	NA	NA	0.2
Other trucks:									
Panels and pickups, 4-tired	268	614				NA		252	1,134
2-axle, 4-tired	201	1,598				NA			1,799
Other combinations:									
2-1	75	NA	NA	NA	NA	NA	NA	NA	75
2-2	78	NA	NA	NA	NA	NA	NA	NA	78
2-S2-2	NA	53	NA	NA	NA	NA	NA	NA	53
3-S1-1	NA	54	NA	NA	NA	NA	NA	NA	54
3-S3-2	NA	55	NA	NA	NA	NA	NA	NA	55
TOTAL MEASURED	25,096	91,802	2,674	13,963	14,536	4,022	1,010	2,165	155,268
7 vehicle classes	24,474	89,428	2,674	13,963	14,536	4,022	1,010	1,913	152,020
13.6 and over	173	181	22	8	1	52		6	443
Percent 13.6 and over	0.7	0.2	0.8	0.1		1.3		0.3	0.3

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(1) *The Freight's the Weight*, by M. F. Kent, Highway Research Board Proceedings, 37th annual meeting, 1958, vol. 37, pp. 21–44.

(2) *Line Haul Trucking Costs in Relation to Vehicle Gross Weights*, report of Committee on Economics of Motor Vehicle Size and Weight, Highway Research Board Bulletin 301, 1961.

(3) *Commercial Motor Vehicle Nomenclature*, Society of Automotive Engineers, Inc., 1961 SAE Handbook, pp. 776–777.

(4) *Trailer Manufacturing Leader Notes Trend Toward Larger Truck Trailers*, by Fred Neumann, Traffic World, vol. 112, No. 7, Nov. 17, 1962, pp. 31–32.

APPENDIX I

Legal Maximum Semitrailer Lengths and Possible Permitted Lengths

The following data show the legal maximum length of semitrailers and the possible permitted lengths[1] calculated from the legal lengths for tractor semitrailers in the Continental United States.[2]

Number of States and maximum legal length limits for semitrailers as of May 1, 1957 and July 1, 1962 were,[3,4] as follows:

Number of States			*Semitrailer length, feet*
1957	*1962*		
15	1	(W. Va.)	35
1	1	(Ga.)	39½
6	12	. . . . .	40
1	1	. . . . .	42
3	1	. . . . .	45
0	1	. . . . .	50
1	1	. . . . .	55

The following list shows the number of States that prescribed length limits on the tractor semitrailer, and the calculated maximum possible for semitrailers.

Number of States		*Tractor semitrailer length, feet*[5]	*Calculated semitrailer length, feet*
1957	*1962*		
2	0	. . . . . 45	38
1	0	. . . . . 48	41
13	17	. . . . . 50	43
0	5	. . . . . 55	48
4	6	. . . . . 60	53
1	2	. . . . . 65	58

As shown, in 1957 semitrailers 40 feet long were permitted in six States, and by 1962 twelve States permitted semitrailers of this length. Semitrailers more than 40 feet long were permissible in 25 States in 1957 and in 35 States by 1962.

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[1] In States where there are no restrictions on length of semitrailers the maximum possible length (van bodies) was assumed to be 7 feet less than the permitted tractor semitrailer combination length. Automobile transport bodies may exceed these lengths when an automobile is carried above the tractor cab, a practice that is permitted in most States.

[2] Includes District of Columbia, making 49.

[3] From *Summary of Size and Weight Limits and Reciprocity Authority (By Regions), in Effect as of May 1, 1957*, American Trucking Assoc., Inc.

[4] From *Summary of Size and Weight Limits and Reciprocity Authority (By Regions) in Effect as of July 1, 1963*, American Trucking Assoc., Inc.

[5] Nevada had no restrictions on length of tractor semitrailer.

Summary Analysis of Reports on State Highway Department Management

BY THE ECONOMIC RESEARCH DIVISION
BUREAU OF PUBLIC ROADS

by **PRISCILLA FAMOUS, Management Analyst,**
Administrative Research Branch

No single type of organization structure is best for all State highway departments, according to the conclusion reached from an analysis of 76 management reports summarized in this article. However, the information developed from these reports, including notation of the types of structural organizations most often inefficient, should be of interest to those concerned with directing and managing the organization of State highway departments. Interest in these organizational structures has developed as a result of the attention being focused on the necessity for a transportation system adequate to promote and meet the needs of economic growth, and for the wisest possible use of the large sums of money being appropriated for highway construction.

This summary analysis of the findings and recommendations of consultants, committees, and others engaged to survey the management of State highway departments should be useful as a guide for highway administrators in evaluating their organizations and highway researchers in outlining further studies in this area.

Introduction

THE EXPANSION of highway operations in recent years, and the accompanying increases in personnel and amounts of money spent for highway activities, have focused the attention of highway administrators on the necessity of applying to highway administration the best management practices and procedures available. Many of the State highway departments have made studies concerning their organizational and management needs. This article, an analysis of the resultant reports, serves to determine the problem areas within the highway organizations and to indicate those areas most often inefficient. This analysis of the findings and recommendations made to improve the areas examined will be useful as a guide to highway administrators in appraising their own organizations and to highway researchers in outlining a program of research studies.

The material presented here is primarily concerned with the structural organization for highway administration. The functional areas of highway operations are not discussed except when necessary to emphasize the structural concept. Recommendations as to the duties and responsibilities of either the organizational units or the assigned personnel are not discussed except in general terms.

In addition to reports exclusively on management, analysis also was made of studies or surveys concerned with: (1) highway needs, engineering, and financing; (2) State government reorganization; and (3) reorganization of the executive offices of State government. Studies made by private and civic organizations interested in highway affairs were also included in the analysis. Any relevant discussions of State highway operations in the reports examined have been included. The findings presented here represent information and recommendations from 76 selected reports [1] involving 41 States and the District of Columbia.[2] Not all of the problem areas discussed are covered in each of the reports.

All but two—one in 1935 and one in 1941—of the 76 studies analyzed were made during the period 1947–62: 11 in 1952; 10 each in 1948 and 1950; 7 in 1954; 6 in 1953: 5 each in 1949, 1955, and 1956; and 1 to 4 in each of the other years.

Fifty-five of the studies were authorized by legislative enactment, resolution, or request; eight were made at the request of the respective highway authorities; and five were requested by the governor of the State surveyed. The authority for making the remaining studies is not clear. The surveys were made or directed by legislative investigative committees and commissions, State universities, management consultants, and some by the highway authority. Most of the reports analyzed are in the library of the Bureau of Public Roads; those not in the Bureau's library are in the libraries of the Highway Research Board, National Highway Users Conference, or the Library of Congress.

Reasons

The reasons, as stated in the reports, for making or having the surveys made are nearly as varied as the number of studies represented. Some of the broader reasons given were: to establish highway needs; to investigate relationships between the State and subdivisions; to remove the division of highways from the State department of public works; to strengthen basic organization of highway department; to establish a true highway department in law; and to meet present highway problems.

More specific reasons for recommending changes in the organization include: to separate policy making and administrative functions, to assure continuity in policy, administration, and employment, and thus stabilize the organization; to eliminate duplication of operations and overlapping jurisdictions; to require the commission and its engineering organization to function on a State-wide basis; to improve performance in the area of construction and maintenance; to establish definite lines of authority; to permit firm direction; to give the highway administrative organization constitutional status in order to bar the legislature from making radical changes; to prevent the governor from changing entire complexion of commission; to eliminate injection of politics into highway programs; to meet standards for good administration; and others of a similar expression.

No investigation was made in the analysis presented here to determine whether any of the recommendations presented by the reports have been adopted either by law or administrative action. Highway organization and management investigations, in some States, are a continuous operation. In addition,

[1] For listing of reports examined see pp. 290-292.
[2] For the purposes of this analysis the District of Columbia has been treated as a State.

at the present time (November 1963), eight State highway departments have underway research programs approved by the Bureau of Public Roads, Office of Research and Development, concerning either the organization of the highway department or one of the management operations such as concurrent audit, personnel, performance budgeting, training, and the critical path method of scheduling operations.

Summary

The criticisms and recommendations made in the management reports analyzed tend to support the opinion of many that no one type of organizational structure is best suited for State highway administration. On the basis of the studies analyzed, a conclusion has been reached that the dominant trend recommended for highway organization is for the administrative commission form of management to be modified or replaced entirely by a policy-making commission or an advisory commission, each with a chief administrative officer who is not a member of the commission. The analysis also indicates that management consultants are favoring the organization of highway departments under two major staff units or functional areas—administrative (nontechnical) and engineering (technical). Other recommendations point up the need for tightening the organization by eliminating units not directly related to highways, integrating or combining separate but related functions, and placing authority for the operations of the highway department in the hands of responsible personnel.

Top Administrative Organization

Some discussion of the top or upper echelon of the organizational structure of 31 State highway agencies was included in 51 of the 76 management studies investigated. The changes recommended are treated in the following paragraphs.

Name of highway agency

It was recommended by 16 studies or reports that the name or title of the State agency administering highway affairs be changed. Some of the title changes reflect recommendations for change in the type of administrative organization, such as removal of the highway function from a department of public works and creation of a department of highways. Other changes appear to have been recommended only to indicate a modification of the existing organization or to establish in law a department of highways as distinguished from the State highway commission and thus separate the policy-making group from the operating organization. Recommended title changes are listed in table 1.

Form of organization

Of the 51 reports concerned with top management, 37 suggested a change in the form or type of highway department organization. Eighteen reports proposed the establishment of a policy-making body with an administrator or director to carry out the policy. One of the reports, as an alternative to the appointment of a policy-making body, suggested that a single executive make the policy and that it be administered by the chief engineer. Six of the reports suggested that the activities of the established body be limited by law to policy making or that the policy-making and administrative responsibilities of the administrator be fixed by law.

Eleven reports proposed the establishment of an advisory board or commission to advise the administrator or director of highway activities. Two of these reports suggested that the division of duties and responsibilities between the board or commission and the administrative officer be set forth in law, and one recommended that the board be advisory in "the strict sense of the law." Five of the reports suggested that the advisory board be in addition to the single-executive type of organization already established. Another report recommended two advisory units—one for traffic and one for parking. Three reports suggested that the highway organization be of the commission-director type, but did not specify whether the body was to be policy-making or advisory. Two reports recommended that the administrative type of commission be abolished and a single executive be appointed to administer highway affairs.

One report proposed that the department be placed within the Department of Public Works; and another that the Division of Highways be removed from the Department of Public Works and be given departmental status in the State, and that the advisory board, already provided for by law but to which appointments have not been made in recent years, be reactivated.

Of the remaining 51 reports, 10 recommended that no change be made in the type of existing organization, and four made no recommendation on the type of administrative organization. Although no changes were recommended, three of these reports suggested that the highway commission should be charged by law with policy-making functions and not have administrative or operating functions; and one report suggested that the requirement for highway commission members to serve full time, which in reality they did not do, be removed from the law.

Responsibility for highway operations in the States at the time of this study was vested in different types of organizations. The head of the highway department in 19 States is a single executive, usually titled State highway commissioner or director, who is both the chief executive and administrative officer. In three of these States the law provides for a commission to advise the single executive on the business of the department.

The other 31 States each have a highway board or commission. In 10 of these States the chairman of the board or commission is the administrative officer; 14 States employ a director, who is neither a member of the board or commission nor chief engineer, to be the administrative head; and in 7 States the chief engineer is the chief administrative officer. The division of authority and responsibility between the board or commission and the administrative officer is usually provided for in the law.

Members on board or commission

Recommendations as to the number of members that should serve on highway boards or commissions were made in 19 of the 51 reports, as follows:

No. of reports	*Members*
2	12
1	10
1	9
1	8
3	7
1	6
4	5
3	3
1	Not to exceed 7
2	"Small number"

Two of the 51 reports suggested that the commission be abolished and a single executive be appointed to administer highway affairs; 16 recommended that no change should be made in the number of members on the highway board or commission; and 14 studies expressed no opinion on the subject.

Selecting officials

Some reference to the method of selecting board and commission members or single executive officers was made in 26 reports. All reports proposed that such appointments be made by the governor. However, ten recommended that the appointments be given some form of senate approval, and one recommended confirmation by the governor's advisory council.

Four reports recommended that commission members be appointed on the basis of district

Table 1.—Recommended title changes

Number of reports	Title of highway agency at time study was made	Recommended title
2	State Highway Department.	Department of Public Works.
1	Department of Highways.	Department of Highways and Traffic.
1	State Highway Advisory Board.	State Highway Commission.
1	State Highway Commission.	Board of Highways.
3	State Highway Commission.	Department of Highways.
1	State Highway Commission.	State Highway Department.
1	State Roads Commission.	Department of State Roads.
1	State Engineering Commission.[1]	Public Works Commission, Division of Highways.
1	State Engineering Commission.[1]	State Highway Department.
1	State Highway and Public Works Commission.	State Highway Commission.
3	Department of Public Works.	Department of Highways.

[1] The members of the State Engineering Commission also composed the membership of the State Highway Commission. Reports proposed the establishment of a new department of State government.

or specified geographical areas; and eight advised appointment at-large, one of these further recommended that no two persons from the same county be members at any one time. Two reports recommended that the appointments be made on a nonpartisan basis; one on a bipartisan basis; one that no more than two members of the three-member advisory body be of the same political party; and in the report that recommended a twelve-member commission appointed at large, it was stated that not more than seven members should be of the same political affiliation. Although two reports did not make specific recommendations, they pointed out that an official's interest should be as broad as his field of responsibility and that, therefore, representation on highway commissions should be on a Statewide, nonpartisan basis.

Tenure of highway officials

The tenure of members of advisory and policy making boards and of single-executive officers was discussed in 25 of the 51 reports in which recommendations were made concerning the top administration of State highway departments. Terms suggested vary from 3 to 10 years, as shown by the following list of number of reports and recommended terms: One, 3 years; two, 4 years; three, 5 years; six, 6 years; one, 8 years; two, 10 years; one, not less than 4 years; one, not less than 6 years; one, at the pleasure of the governor but long enough to give opportunity to become acquainted with work; two, no change in term (term was 2 years in one case, the other 6 years, both staggered); four, not specified in years.

In 19 of the reports that discussed tenure, recommendation was made that the terms of commission members be staggered in order to provide continuity of policy, eliminate political domination, and ensure stability of purpose. In connection with tenure, six reports made recommendations regarding the removal of highway officials, as follows: Three, by governor for cause; one, for cause after hearing; one, for cause after determination by district court; and one, subject to removal for cause.

Chief Administrative Officer

Of the 76 reports reviewed 28 made recommendations concerning the chief administrative officer of the respective State highway agencies. Different titles are assigned to the officer charged with the administration of highway affairs, and no particular title can be associated with a specific type of organization. The following list shows the distribution of titles recommended in these 28 reports; a recommended title did not necessarily mean a change in title: 15, Director of Highways, Director of Highway Department, Highway Director, or Director; 4, Commissioner of Highways; 1, Superintendent of Public Roads; 1, Director of Road Policy Board; 2, State Highway Engineer; 2, Chief Engineer; and 3, title not specified.

Where the title for the administrator was not specified, one report suggested that the official be someone other than the chairman of the road board; one suggested that the administrative authority not rest in the chief engineer, whose principal responsibility was to direct operations, but that the secretary of the commission be named administrative officer or that the office of administrator be created; and one specified that the appointee be a "top flight administrator." When the State highway engineer was recommended as the chief administrative officer, one report also recommended that his authority be redefined so that his office would be the clearing house for all matters below the policy-making board.

Further recommendations concerning the chief administrative officer were made in three reports: one, that the director be a member of the road policy board and the only member to work full time; another, that the director of the highway department also be the chairman of the State highway commission; and the third, that something be done to remove the director from the area of policy making where by law he has the right to vote in a tie upon matters coming before the commission.

Method of selection

Selection and appointment of the chief administrative officer usually is within the authority of the commission or the governor. Although specific appointing authority was not suggested in all of the 28 reports, except for two some indication was made as to where the authority should be. The following list shows the appointing authority suggested or the location of the authority indicated and the number of reports showing each: Commission, 13; governor, 8; commissioner or governor, 1; should be responsible to the commission, 3; should serve directly under the board, 1; and not specified, 2.

Three reports suggested qualifying the authority of the governor to select the administrator: One by requiring that the appointment be made with the consent of the senate; another, by requiring that the appointment be confirmed by the council; and the third by requiring confirmation of the appointment by the senate. Two reports suggested that appointing authority of the commission be qualified by requiring the appointment to be made with the approval of the governor.

Tenure

The tenure of the chief administrative officer was not a subject of major concern to the investigators making the surveys. Of the 28 reports only two recommended a term in years; the others made no reference to term or left the question of tenure with the appointing authority. Tenure was suggested, as follows: 4 years, 2 reports; at the pleasure of the commission, 7 reports; at the pleasure of the governor, 4 reports; and none specified, 15 reports. One of the reports, which recommended a 4-year term, specified that the appointee be a "career" official.

Highway Organization at the Staff Level

The data with respect to the staff level of highway organizations is varied.[3] Some reports recommended a general reorganization of the department, others the addition of a major unit, and still others the removal or consolidation of units. All significant recommendations made in the 43 reports that are concerned with organization or reorganization at the staff level are included under the subsequent headings. There are 33 States represented by the 43 reports.

Reasons for changes

The reasons for changes in staff level organization are many, and have, from the benefit hoped to be achieved, been grouped under five headings. Sometimes a report gave more than one reason. Of the 43 reports, 35 gave reasons as follow for making recommended changes in organization: Establish specific responsibility, 15 reports; consolidate like activities, 11 reports; make orderly improvement in the administration of State and/or highway affairs, 12 reports; remove from the highway department activities unrelated to highways, 2 reports; and minimize political influence, 1 report.

Under the broad heading, "establish specific responsibility," are such terms as: "fix lines of authority," "permit firm direction," "separate policy making from administration," and "relieve chief engineer of administrative duties;" under the heading, "consolidate like activities," are "reduce reporting heads," "eliminate waste," "strengthen control," and "achieve efficient highway administration." Under "make orderly improvement in administration, etc.," are such reasons as "modernize administration organization," "take care of expanding responsibilities," "better management for future programs," and "provide continuity of policy."

Organizational structure

Of the 43 reports 27 made recommendations concerning the number of staff units or divisions into which the highway department should be organized for the purpose of administering highway activities. The number of divisions recommended are as follows: Two, 16 reports; three, 2 reports; four, 2 reports; five, 4 reports; six, 1 report; seven, 1 report; and nine, 1 report. As indicated, 16 reports suggested that all highway activities be divided into two principal staff divisions—administrative (nonengineering) and engineering (technical). Thirteen of these reports provided that each of the two units be headed by a person responsible to the chief administrator. The other three provided that the engineering or technical unit be headed by an engineer responsible to the chief

[3] The term staff level as applied here means that level of organization directly below the chief administrator. The different staff units are termed divisions, although many reports following the practice of the State concerned in giving a name to the units used other titles, such as department, bureau, and section.

administrator, and that the administrative or nontechnical functions be handled directly by the chief administrator. Several of the recommendations made to set up two main staff divisions, of necessity, included the creation of the nontechnical office and the appointment of a manager or director.

Two reports in which three divisions were suggested called for an administrative or management services division and engineering division, and a third division, entitled in one "Traffic Services," and in the other "Planning and Programming." An official responsible to the chief administrator was recommended to head each of these divisions.

Two reports recommended an organization made up of four divisions, one to be an engineering or technical division. One report suggested the three remaining divisions should be for accounting, motor vehicles, and public safety; and the other report suggested divisions of administrative services, planning, and maintenance.

Four reports recommended that the headquarters office of the highway department consist of five divisions; as shown for each report: (1) administrative, design, construction and maintenance, planning and research, and field operations; (2) administrative, planning, technical, maintenance, and tourist bureau; (3) program, operations, accounting, personnel, and purchasing; (4) administration, planning and research, operations, engineering and bridge design, and construction. Some of these titles refer to the same function and could be used interchangeably. For the purpose of this analysis, however, the same titles given in the report analyzed have been used.

One report recommended six organizational or staff divisions, as follows: Engineering, personnel, motor vehicles, comptroller, public safety, and publication (for publishing the department's magazine). The two reports suggesting seven and nine divisions, respectively, were for States where the highway function is incorporated within a State bureau or department of public works. One report recommended that there be established, within the public works bureau, a division of highways that would include all engineering activity divided into seven major sections of—design, construction, maintenance, local roads, traffic operations, planning, and right-of-way—the administrative or nontechnical activities would remain a part of the overall administrative authority of the public works agency. The other report recommended that the existing division of highways in the public works department be divided into eight bureaus, plus a unit headed by a State-aid engineer. The eight bureaus suggested are: Location and design, bridges, construction, maintenance, traffic, materials, right-of-way, and planning. No recommendation was made concerning administrative matters.

New divisions

The establishment or creation of new divisions was recommended by 27 of the 43 reports in which recommendations were made with respect to highway organization at the division or staff level. When the report suggested that a present division be expanded, either by the addition of personnel or functions, or that the principal functions be reorganized under a new title, these divisions were not included in this summary analysis—only those divisions suggested for establishment or creation as new organizational structures have been included. New divisions to be established or created and the number of reports suggesting them are, as follows.

Urban, 4; State-local, division of local government, or similar title, 7; traffic engineering, 3; personnel, 5; advance planning and programming, 2; construction, 2; comptroller or fiscal, 2; land, 1; planning and research, 2; planning and administrative research, 1; public information, 6; public and intergovernmental relations, 1; and special projects (tolls), 1. Number of reports listed is greater than 27 because some reports made more than one recommendation. In addition, four reports recommended that a legal division be established in the highway department in cooperation with the State's attorney general; four others recommended that a full-time legal staff be provided; and another recommended that an office of legal services be established.

Other organization recommendations

In addition to the new divisions suggested the reports recommended other organizational changes. Six reports proposed that units existing in the highway organization at the section or lower level be raised in status to division level. Recommendations that the function be given division status were made for: Traffic engineering, 2 reports; planning, 2 reports; traffic and safety, 1 report; and State-aid construction, 1 report. One of these reports also recommended that the office of the State highway patrol and the bureau of motor vehicles, provided that the functions remained under the direction of the highway administrator, be given the same status level as the heads of the recommended division of engineering and business management.

One report recommended combining the divisions of materials and tests, and planning and design, thus making two divisions from four. Another report recommended the reduction of the number of divisions in the department to the smallest possible number required to keep the span of control at an efficient minimum.

Eight reports suggested that divisions not directly related to the operations of the department be removed from the highway organization, and that these divisions be abolished, be made a separate agency of the State government, or be included in another established agency. Typical recommendations were: Transfer safety division to State department of traffic and transportation; integrate purchasing with purchasing division of State department of administration; remove motor vehicle division except for collection of fees; and abolish traffic safety division.

Eight reports made 10 recommendations for the creation of a unit below the division level. Units suggested for creation and the number of recommendations for each are as follows: Personnel, 1; public information, 2; local road section, 1; program planning section in planning and research division, 1; section to prepare and maintain a long-range program of improvements, 1; advance programming and planning section in the planning division, 1; research section apart from materials, 1; section to aid counties and municipalities in connection with Federal-aid, 1; and section or unit directly under the State highway engineer to study operations, methods, procedures, and work functions, and recommend changes, 1.

Organization in the Field

In this analysis the field refers to offices other than those of the central or headquarters offices of the highway departments. Such organizational units will be called districts, although in the management reports analyzed the terms district, division, and section were used. Most of the recommendations as to organization at the district level have been given little emphasis in the overall management discussions.

Out of 76 reports reviewed, 20 reports suggested changes or modifications in field organizations, 9 of these reports made recommendations as to the number of districts necessary for the proper handling of highway affairs. Number of districts recommended are, as follows: Five, 1 report; six, 3 reports; not more than nine, 1 report; six to eight, 1 report; reduce to unspecified number, 2 reports; and increase to unspecified number, 1 report.

One State, for which three reports were examined, had offices termed zones in its organization concerned with field work in location and design, and right-of-way acquisition. Little relationship existed between these zones and the district offices. One of the reports suggested that there be no change in these areas, but that they be called regions rather than zones. Another suggested that the zone offices be discontinued and the duties and responsibilities be taken over by the district engineers. The third report, which made no reference to the zones, suggested that the number of district offices be reexamined but made no recommendation as to the number needed to do the work.

Two reports recommended that in the field organization the duties of district engineer should be confined to maintenance, and that he be made responsible to the chief maintenance engineer. Each of the following recommendations was made in a single report.

- The structure of field operations should more properly be left to administrative determination.
- Revise district organization to include all construction and maintenance activities under the head of district engineer.
- Organize district with district engineer and two assistant engineers, one for construction the other for maintenance.
- District engineers should be under the technical direction of the chief engineer.
- Establish each district as a complete administrative unit headed by a district engineer,

hav[illegible] assistant district engineer to head each major function, and establish direct line of authority from chief of operations to each district engineer.

- Formulate policies and programs in the central office; the job of the districts is to carry them out.
- Give more authority to district engineer thereby reducing the amount of detail reaching central office division heads.
- Channel maintenance policies, controls, and operation directly through the district engineers; and require that district maintenance engineers report to the district engineer.
- Locate district offices on a permanent basis. All major functions of the department should be directly administered in each district under firm policies established by top administrative staff members.

Table 2.—Recommendations on personnel practices

Number of reports	Recommendations
	CIVIL SERVICE OR MERIT SYSTEM
8	Establish a merit system.
3	Establish a civil service system.
3	Establish a personnel system and/or program.
4	That, until a State-wide merit system is established a merit system be set up within the highway department by administrative action.
1	Amend civil service law to include all positions in highway department.
	RECRUITMENT AND TRAINING
13	Institute an organized system of recruitment.
11	Make available or increase in-service training.
5	Establish training project with engineering schools for in-service training and as a means of interesting undergraduates in highway work.
	SALARY AND WAGES
2	Remove constitutional or statutory ceilings on salary of public officers.
2	Raise salary of chief administrative officer.
10	Compensate at salary level effective in private enterprise and other competitive areas (other State highway departments).
5	Provide salary increases for all engineers, and technical and professional personnel.
2	Adopt policy under which salary and wage rates will be adjusted to a cost of living index or other base.

Personnel Practices

Recommendations on personnel policies and practices were made in 37 reports of the 76 reports analyzed. These recommendations do not include those to establish personnel divisions, which were reported in the discussion of highway department organizational structure. Seven reports recommended that laws be enacted to establish a State retirement system. The other suggested changes and practices are shown in table 2.

The suggestion was made in one report that the number of positions secondary to principal division or department heads be increased so that experienced, capable persons would be available for the higher positions as normal retirements occur.

Two reports discussed the appointment and tenure of division heads. One recommended that the law giving the administrative body authority to appoint assistant highway engineers be repealed and that the authority be given to the State highway engineer. The other recommended that division heas dserve according to civil service regulations rather than at the pleasure of the director of highways.

Three reports suggested that personnel practices be removed from the influence of politics. One of these recommended that the selection, promotion, and treatment of all employees be on a nonpolitical basis; another report suggested that restrictions be placed on political activity of officers and employees; and the third report stated that the only permanent solution to personnel problems is to take the highway department out of politics.

REPORTS ANALYZED

The reports reviewed and included in the summary analysis of this article are listed in the following paragraphs.

Alabama:

The Reorganization of Alabama's State Government, Legislative Reference Service, 1950.

Arizona:

Report of the Arizona Special Legislative Highway Study Committee to the Governor and Legislative Council, December 1953.

A Report to the State of Arizona on Operations of the Highway Department: State Highway Engineer, Motor Vehicle Division, Highway Patrol Division, Traffic Safety Division, and Courts of Limited Jurisdiction, based on studies conducted cooperatively by the American Bar Association; American Association of Motor Vehicle Administrators; Traffic Division, International Association of Chiefs of Police; and Traffic Institute, Northwestern University, 1953.

Arkansas:

An Organizational and Personnel Analysis of the Arkansas State Highway Department, prepared by the University of Arkansas, and published by the Arkansas State Highway Commission, 1952.

California:

California's Highway Problem, A Report by Joint Fact-finding Committee on Highways, Streets and Bridges. Report made to the Fifty-seventh (First Extraordinary) Session of the California Legislature, January 13, 1947.

General Survey California Division of Highways, Report to the Joint Legislative Budget Committee, by Booz, Allen and Hamilton, February 1955.

Colorado:

The Committee Reports to John Q. Public on the Long Range Highway Plan, by the Colorado Highway Planning Committee, 1951.

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State and Local Governmental Relationships in Connecticut, A Report of the Commission to Investigate the Relationship between the State and its Subdivisions, [1955].

Delaware:

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Florida:

Report on a Study of Florida Highways for the State Road Department of Florida, by Parsons, Brinckerhoff, Hall & MacDonald, July 1952.

Report to Legislative Council Covering Florida Primary Roads, by Select Committee on Roads, compiled and published by the Florida Legislative Reference Bureau, September 1954.

Idaho:

Idaho Highways, a report of a study made for the Idaho Highway Study Committee, by Public Administration Service, Chicago, San Francisco, and Washington, December 1949.

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A Highway Improvement Program for Illinois, a report for Illinois Division of Highways by Griffenhagen & Associates, November 1948.

Report of the Illinois Highway and Traffic Problems Commission on A Highway Improvement Program for Illinois, February 1949.

Organization and Functioning of the State Government, a report of the Commission to Study State Government made to the Illinois General Assembly, December 1950.

A Program to Meet Present Illinois Highway Problems, recommended to the 68th General Assembly by the Illinois State Chamber of Commerce, 1953.

Indiana:

Report on State Highway Commission, by the State of Indiana Commission on Organization of the Executive and Administrative Branches of the State Government, October 1952.

Iowa:

Report of Highway Investigation Committee, published by the State of Iowa, November 1948.

Report of the Governmental Reorganization Commission to Governor William S. Beardsley for Submission to the Fifty-fourth General Assembly, 1950.

Iowa Highway Needs 1960–1980, A Plan to Pace Highway Development with Economic Growth, a report to the Iowa Highway Study

Committee by the Automotive Safety Foundation, Washington, D.C., Nov. 1, 1960.

Kansas:

Highway Needs of Kansas: An Engineering Analysis, a report of the Kansas Highways Fact-Finding and Research Committee to Governor Frank C. Carlson, The State Highway Commission, The Legislative Council, and Members-elect of the 1949 Legislature, by the Automotive Safety Foundation, Washington, D.C., Dec. 15, 1948.

General Report of the Legislative Committee on Economy and Efficiency, 1959.

Kentucky:

A Report on Kentucky Highways, Their Administration and Financing, by the Public Administration Service, Chicago, San Francisco, and Washington, October 1947.

Kentucky Highway Systems and the State Department of Highways, prepared by the Public Administration Service for the Kentucky Department of Highways. Published by the Kentucky Legislative Research Commission, Research Publication No. 29, 1951.

A Highway Program for Kentucky, Engineering Recommendations for Adequate Road and Street Systems, by the Automotive Safety Foundation, Washington, D.C., November 1955.

Louisiana:

Louisiana's Highway Problem, An Engineering Analysis for the Louisiana Legislative Council, by the Automotive Safety Foundation, Washington, D.C., September 1954.

Maine:

Maine Highway Needs, An Engineering and Economic Study, prepared for the 94th Legislature by the Maine State Highway Commission in cooperation with the United States Public Roads Administration, March 1949.

A Plan for Highway Classification in Maine, An Engineering Report to the State Highway Commission, by the Automotive Safety Foundation, Washington, D.C., 1952.

Organization and Administration of the Government of the State of Maine, A Survey Report, by the Public Administration Service, Chicago, San Francisco, and Washington, June 1956.

Maryland:

Improving Road Administration in Maryland, a report to the Governor of Maryland by the Commission on State Programs, Organization and Finance, Nov. 15, 1955.

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Report on Organization of Massachusetts Department of Public Works, by Griffenhagen & Associates, Feb. 28, 1948.

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Staff Report, No. 19, to the Michigan Joint Legislative Committee on Reorganization of State Government, Concerning The Michigan State Highway Department, by Arthur W. Bushell and William J. Cox, April 1951.

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A Report on State Reorganization by the Legislative Fact-Finding Committee on Reorganization of State Government, 1950.

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Factual Discussion of Motortruck Operation, Regulation and Taxation (1951). 30 cents.

Federal Role in Highway Safety, House Document No. 93 (1959). 60 cents.

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First Progress Report, House Document No. 106 (1957). 35 cents.

Final Report, Parts I–V, House Document No. 54 (1961). 70 cents.

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The 1961 Interstate System Cost Estimate, House Document No. 49 (1961). 20 cents.

U.S. HIGHWAY MAP

Map of U.S. showing routes of National System of Interstate and Defense Highways, Federal-Aid Primary Highway System, and U.S. Numbered Highway System. Scale 1 inch equals 80 miles. 25 cents.

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Aggregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

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